MANUAL FOR STRUCTURAL ENGINEERING EVALUATION OF EXISTING BUILDINGS FOR SOLAR COLLECTOR INSTALLATION

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CHAPTER 1

USE OF THE MANUAL

USE OF THE MANUAL

1.00 INTRODUCTION

This manual covers the structural engineering analysis of existing buildings to accommodate the installation of new solar collectors on the roofs of these structures.

Solar collectors place additional loads on the roofs of the structures on which they are installed. They add their own weight, they add drifting snow and they increase wind effects. As 90% of PUSH Project solar installations are planned for existing buildings, an Owner immediately faces the question - "will the roof of my building support the new solar collector system?"

Solar projects are extremely sensitive to cost, as payback periods for the installations are generally quite long. Project budgets will seldom tolerate the inclusion of roof strengthening costs. Therefore, an efficient structural analysis is often the difference between acceptance and rejection of a solar panel installation proposal.

The need for this manual is based on a recognition that engineers tend to be conservative both in designing and in checking designs of structures, because of the grave consequences of structural failure. For example, in designing a simple span steel beam, the design span of the beam is usually taken as the distance measured from centreline to centreline of columns. Using the clear span of the beam between centrelines of connectors results in a substantial apparent increase in the beam's capacity.

This manual shows how a design, not apparently meeting current Code requirements based on conventional structural design or analysis procedures, can be re-evaluated using more rigourous techniques, to make it conform to the appropriate standards.

It provides the engineer with techniques enabling him to approve a structure for a solar installation, rather than condemning it.

2.00 SCOPE

The types of buildings covered in this manual are structural steel buildings and of reinforced concrete buildings. Timber structures are not addressed because most older timber structures tend to be well below current Code standards for strength. In addition, the current trend in the Timber Code in reducing allowable stresses in wood may render even previously well designed structures marginal or unacceptable when compared to new standards.

The manual deals with cases of minor overloads (less than 10%) which may occur even after rigourous analysis techniques have been applied. Interpretations of the current building codes are examined which may allow certification of a structure as complying with the intent of the Code if not the letter of the Code.

A discussion for methods of reinforcing substandard structures is not included in this manual.

3.00 APPLICABILITY OF MANUAL

The extra loads imposed on an existing roof by a solar installation generally do not add more than 10% to 15% to current building code design loads for roofs without collectors. The manual is devoted to the analysis of structures which generally conform or almost conform with current design standards.

The age of the structure may not have any bearing on whether it will be able to sustain the increased design loads from a solar collector installation. Older structures, while often designed for lower loadings, have higher redundancy which can be taken advantage of in

reanalysis. Newer structures, while usually designed for more realistic loadings, often have less redundancy and less reserve capacity because of using more advanced design methods.

Structures with large dead load/live load ratios show the most promise for accepting the additional loads imposed by solar collectors. In particular, reinforced concrete structures, originally designed by working stress methods, can show substantial increases in load carrying ability if analyzed by ultimate strength techniques. Steel structures with high dead load/live load ratios also are likely to benefit from reanalysis using Limit States Design techniques.

Light structures (low dead load/live load ratios), long span structures, and structures with no redundancies are less likely to accept even minor additional loads. An example of these types of construction is a roof constructed using open web steel joists and non-cellular roof deck.

4.00 METHODOLOGY

The evaluation of existing buildings for a solar collector installation requires the use of good engineering judgement, modern analysis techniques, rational evaluation of the codes. Loads are discussed in Chapter 2. The first step in the evaluation is a preliminary analysis. It is covered in Chapter 3. The preliminary analysis determines whether the building merits detailed study. Chapter 4 deals with the detailed information gathering of material properties, structure geometry and structure details. The final two chapters concern detailed analysis of reinforced concrete and steel structures. In each of the later chapters of the manual, guidelines are given to help the engineer decide whether or not it is necessary to proceed with the investigation.

The two appendices at the end of the manual summarize historical trends in material properties and in engineering design and construction.

CHAPTER 2

LOADS

1.00 COLLECTOR LOADS

1.01 Dead Load

The dead loads of the solar collector panels, the collector support system and the piping are relatively small. Panels weigh about .15 to .2 kPa (3 to 4 psf) based on collector area. Collector support systems weight .1 to .15 kPa (2 to 3 psf) including piping. Even though they are small, actual weights of the collectors, support systems and piping should be calculated, not 'ball-parked'.

Collector system dead loads are transferred to the roof structure by the legs of the support system. Thus, collector dead loads usually occur as discrete point loads on the roof rather than as uniform loadings. Provision for accommodating these external forces to the building must be made in the building structure.

The dead loads of the collector system are not usually sufficient to resist overturning or sliding between the roof and the collectors caused by wind. Therefore, the installation must be fastened to the roof by connections which resist the associated tensile and shearing forces.

1.02 Snow Loads

Snow accumulates on sloping collector panels at least temporarily, even when the collector angle is as high as 60° to the horizontal. With some types of collector, the snow accumulation may cause bending or flexing of the panel. Connections between the panel and the support frame should be designed accounting for such flexing.

Often the temporary accumulation of snow on a solar panel slides off and creates a pile on the roof at the bottom of the collectors. This pile of snow has two effects on the building supporting the solar assembly:

a) A concentrated load of snow builds up on the roof due to the pile of snow.

b) The snow pile may restrict wind blowing under the collectors. Without the snow pile, wind blowing under the collectors would increase in velocity and scour the snow in the vicinity of the collector which would normally accumulate on the roof. An accumulation of sliding snow piled at the base of the collector may therefore result in the increased drifing of snow behind collector assemblies.

Recommended snow loads for the design of the collector panels as well as the collector support system are covered in detail in section 2.02 of this chapter.

1.03 Wind Loads

Wind pressures are significant in the design of solar collector support assemblies. Collectors act like billboards in the wind. They are small and light and therefore more subject to the effects of gusts than more massive structures. Gust factors associated with solar collector support systems are increased from 2.0 to 2.5 to account for this.

Wind forces on a collector system are generally significant enough that the assembly must be tied down to prevent its overturning or sliding. Wind loads are transferred to the building structure by the legs of the collector supports. The loads which occur at each collector leg have both vertical and horizontal components of force. The horizontal forces are generally transferred into the roof diaphragm system, from which they are transferred into the lateral force resisting system of the building. The vertical forces may act either upwards (tension) or downwards (compression) depending on the wind direction. Therefore, the collector system must be anchored to the roof structure into such elements as have adequate member resistance or dead load to prevent uplift of the collector array.

Specific wind pressures acting on collector assemblies are covered in section 2.03 of this chapter.

2.00 BUILDING LOADS

2.01 Dead Loads

Accuracy

In calculating the dead loads of a building structure, it is important to be realistic, but not overly conservative.

The dead load is usually made up of a series of components such as roofing, structural steel, metal deck, and general allowances. General allowances for ductwork, piping, sprinklers, lighting and ceilings may also be included. The dead loads can be evaluated more accurately after the structure has been built than they could have been during the design stage before the structure was constructed. Table 1 shows a typical comparison between dead loads estimated at the design stage of a building construction project and actual dead loads calculated after a structure is complete.

TABLE 1 - COMPARISON OF PRELIMINARY DESIGN ESTIMATES TO ACTUAL WEIGHTS

Dead Loads	Preliminary Design	Actual
roofing	.50 kPa	.50 kPa
structure	.50 kPa	.24 kPa
lighting	.15 kPa	.09 kPa
mechanical	.25 kPa	.20 kPa
ceiling	.10 kPa	.09 kPa
Sub-Total	1.50 kPa	1.12 kPa
Live Loads		
snow	2.40 kPa	2.40 kPa
TOTAL LOAD	3.90 kPa	3.52 kPa

In this case the estimated design dead load is about 25% conservative. With a total load (dead and live) on the structure of 3.9 kPa a 25% reduction in the dead load translates into about an 10% reduction in the total load. Previously it was noted that solar collectors may only add 10% to 15% additional loading. Therefore, a careful evaluation of actual dead loads versus original design assumptions may be sufficient to allow approval of a structure for solar installation without the need to even resort to further structural analysis.

Roofing

A major factor to be considered in evaluating dead loads is the type of roofing system installed. Is the roofing a conventional type of roofing, or is it a heavily ballasted inverted roofing system? It is equally important to determine whether the present roofing is the original roofing on the building. Roofs eventually require replacement, usually within 10 to 25 years of their original installation. If the original roofing has been replaced or if it has been overlaid with a conventional membrane or an inverted system, it is important to determine the new roofing weight.

A conventional roofing system is generally quite light, weighing in the order of .2 to .5 kPa (including insulation). An inverted roofing system, sometimes called an "upside down roof" or a "protected membrane roof", weighs in the order of .8 to 1.2 kPa. If the roofing on a building has been replaced and has been changed from a conventional system to an inverted system, reserve capacity in the structure will have been used up. Some may yet remain but it is doubtful.

How does one determine a realistic roofing weight? When the roof construction is known, the most common method of obtaining the roof weight is to calculate the weight of each component. For example, Table 2 shows the calculated dead weight of a conventional 4-ply membrane roof with 50 mm of fibreglass insulation and a vapour barrier:

TABLE 2 - CONVENTIONAL ROOFING SYSTEM

	UNIT WEIGHTS*	
vapour barrier	.007 kPa	(15 lbs/square)
50 mm fiberglass	.096 kPa	(200 lbs/square)
4 plies #15 felt	.029 kPa	(60 lbs/square)
3 mop coats	.029 kPa	(60 lbs/square)
1 top mop coat	.012 kPa	(25 lbs/square)
1 flood coat	.029 kPa	(60 lbs/square)
gravel	.294 kPa	(600 lbs/square)
	.496 kPa	(1020 lbs/square or 10.2 psf)

* per CRCA recommendations for SI-51 Asphalt Hot Process Class A roof.(2)

Table 3 shows the calculated dead weight of an inverted roof system.

TABLE 2 - INVERTED ROOFING SYSTEM

	UNIT WEIGHTS*	
13 mm gypsum board	.096 kPa	(200 lbs/square)
4 plies #15 felt	.029 kPa	(60 lbs/square)
3 mop coats	.029 kPa	(60 lbs/square)
top coat	.012 kPa	(25 lbs/square)
flood coat	.029 kPa	(60 lbs/square)
3" rigid polystyrene insulation	.030	(63 lbs/square)
3" gravel ballast	.718 kPa	(1500 lbs/square)
	.943 kPa	(1960 lbs/square or 19.6 psf)

* per CRCA recommendations for S-58 Asphalt Hot Process Class A roof.(2)

Avoid the temptation of using standard unit weights of roofing systems published in handbooks, such as the CISC Steel Handbook(3), instead of adding up the components. A conventional 4-ply built-up roof with 10 mm of gravel ballast has a standard unit weight from a handbook of .366 kPa and the insulatin weighs .035 kPa for a total unit weight of .401 kPa. This compares unfavourably with the derived unit weight of .496 kPa.

If the roof construction in unknown, a cut test may be necessary. This is the most accurate method of determining roofing weight. Cut and weigh a sample of the roofing, including gravel, membrane, insulation and vapour barrier or underlay. This may sound complicated and expensive but is not! If one has no idea of the type of roof construction, consider the consequences of conservatively estimating the weight of the roofing to save a $100.00 cut test. It could cost the project!

Structural Elements

Structure weights can usually be determined quite accurately when carrying out a reanalysis. The structural engineer knows the components of the system and their weights and he does not have to estimate their sizes as is necessary in the design process.

To accurately assess the structure's dead weight, it is important to measure all structural elements accurately. This includes concrete thickness, structural steel beam joist dimensions, metal deck profiles, bridging, bracing, etc.

Again it is important to avoid using standard unit weights of structures, such as structural steel allowance - .025 kPa (5 psf). Calculate the weight of all components of the structure. Take note of the additive nature of the load or the load path of the weights. For example, the structural beams support the dead weights of the roof deck, not vice versa. The deck may support the ceiling if the ceiling hangers are attached to the deck rather than the beams or joists. Often, however, the ceiling is attached only to the structural steel. Examination of the load path may be critical in the analysis of some of the structural elements, in this case, the capacity of the deck.

Allowances

Common dead load items which are often conservatively assessed during the design stage are allowances for mechanical elements, allowances for electrical items such as lighting and allowances for both floor and ceiling finishes. When assessing an existing structure, there is

no need for inaccuracy or conservatism. It is possible to measure ductwork, obtain mechanical equipment weights, count light fixtures and calculate quite precisely the weight of ceilings and floor coverings. There may or may not be some reserve capacity in the original design loads for these items, depending on how conservative the original designer was.

A question which often arises when evaluating mechanical, electrical and finishing weights is whether or not to include for future installations. Indeed the same type of question arises concerning the roofing -- Should I allow for future roof replacement? The answer to the question is not a simple yes or no! It really depends on the priorities of the Owner. But if the solar project is to be economically feasible, the structural engineer cannot allow for reserve capacity in the structure for some future indeterminate event. It is unlikely the Owner will be able to have both reserve capacity for future installations as well as a solar installation without structural reinforcement.

2.02 SNOW LOADS

A primary load change which occurs to a building structure as a result of the installation of solar collectors is an increase in the snow load. The collectors shelter the roof to some extent and can cause additional drifting snow load.

PWC Briefing Document S2 Structural Design for Solar Collectors gives recommended snow loads for roofs subjected to the influence of solar collectors. This document defines a critical clearance under the collectors. For collectors set above the critical height, the influence of the collectors is to uniformly increase the snow load by 10%. Below the critical height, drifting and sliding snow tends to accumulate in a sawtooth pattern around the collector. The magnitude of the change in snow load is therefore dependent on the collector assembly's height, its roof clearance. It also depends on the number of rows in the array.

The PWC Briefing Document S2 represented state of the art (or science) at the time it was written in 1980 and 1981. At that time, very little field data or experimental information was available on which to base the recommendations. Recent research ([4]) ([5]) ([6]) using wind tunnel and waterflume techniques and two years of field data ([7]) have increased the body of knowledge concerning snow accumulation in the vicinity of solar collectors and the document is therefore somewhat out of date. The experimental work suggests that with adequate clearance beneath the solar collectors, no additional snow will accumulate on the roof. But the actual clearance required is still under discussion and undetermined.

For the purposes of this manual, the Briefing Document S2 will be used as the basis for assigning snow loads.

2.03 WIND LOADS

The wind loads on solar collector assemblies outlined in Section 1.03 must be transferred into the structure of the building. It is common practice where possible, to transfer these loads into the structure at column locations. This avoids causing negative bending moments in the roof structure. It also saves all reserve flexural capacity in the roof structure for increased snow loads.

Wind loads transmitted from the collector installation are generally small compared with other forces in the building structure. They are generally easily accommodated. The horizontal component of the load can be transferred into the roof diaphragm system and from there into the building's lateral force resisting system. Vertical loads can be accommodated by the building columns. No indirect wind effects of consequence on the parent building have been noted with solar installations. That is, the wind patterns on the building itself do not change enough to require re-evaluation of the effects of wind on the building as a whole. Only the solar collector wind loads need to be added to the structure at the roof level.

PWC Briefing Document S2 Structural Design for Solar Collectors will be used as the basis for assigning wind loads to the collectors. Recent research (8) (9) indicates that a uniform set of co-efficients, as set out in Document S2, for both tall and low buildings may be an over generalization and co-efficients for collectors mounted on sloping roofs should differ from those for collectors mounted on flat roofs. Therefore, as for snow, the document may be somewhat out of date.

3.00 REFERENCES

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2. Canadian Roofing Contractors' Association Specification Manual.

3. Handbook of Steel Construction, Third Edition, 1980, Canadian Institute of Steel Construction.

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CHAPTER 3

PRELIMINARY ASSESSMENT

PRELIMINARY ASSESSMENT

1.00 INTRODUCTION

The preliminary assessment is the first stage of the stepped analysis for evaluating a building structure for the installation of solar collectors. The purpose of the preliminary assessment is to determine the feasibility of the installation before spending too much of the Owner's money.

At this stage no material testing is done. It is important however, that the material property assumptions and analyses carried out at this stage not be unduly conservative. Realistic material property assumptions, good analysis techniques and lower bound estimates of existing loadings will yield analytical results which truly indicate whether further detailed analysis is warranted and whether or not the project is likely to be able to proceed without structural reinforcement. Conservative estimates of material properties, crude analyses or overestimation of existing loadings may result in condemning the building as being unable to accommodate the solar installation.

Reinforced concrete structures can usually sustain additional loads. Heavy structural steel buildings have an average chance of being able to sustain additional loads. Light steel joist or steel truss structures usually have little reserve capacity and will not accept solar installations without being overstressed.

2.00 DRAWINGS

It is helpful to obtain drawings of the structure at this stage, if at all possible. In order of preference, the following types of drawings are ranked from most useful to least useful:

(a) structural as-built drawings
(b) structural design or contract drawings

(c) architectural as-built or design drawings
(d) other drawings (preferably mechanical)

Structural as-built drawings are rare even for recently constructed buildings. Mechanical and electrical as-builts are quite common. Structural design drawings are relatively common for buildings constructed in the last 30 years. Architectural drawings are generally available for most buildings constructed.

From structural design or as-built drawings, the following important original design criteria may be obtained:

(a) Approximate date of construction - refer to title block.
(b) Building Codes in effect during design - refer to general notes.
(c) Design loads - on more recently constructed buildings.
(d) Material properties, ie. - concrete strength
- reinforcing steel yield strength*
- structural steel yield strength or allowable stress used in design.
(e) The structural system.

*Note: a) structural grade reinforcement fy = 227 MPa
b) intermediate grade reinforcement fy = 276 MPa
c) hard grade reinforcement fy = 344 MPa

3.00 SITE

3.01 General Observations

During the initial site reconnaissance, try to obtain a general overall impression of the condition of the building and the integrity of the structure. If design or as-built drawings are available, the structural system and its components should be compared with the drawings.

What has changed since the building was originally built? Has new roofing been installed? Has new mechanical equipment been added -

chillers, condensers, etc.? Has the interior been renovated with new ceilings, floors or partitions?

On the roof, the liklihood of snow drifting should be investigated. Snow shadow areas adjacent penthouses, equipment or parapet walls may modify the design snow load conditions for the roof from a uniform snow load co-efficient of Cs = .6 or .8 to Cs = 2.0 to 3.0. Sufficient measurements should be taken to define or delineate these areas. Specific requirements for snow build-up in wind sheltered areas were first introduced into the National Building Code in 1960. If the building is of an age in which these snow shadow were taken into account in the original design, they are possible locations for collector installations as the roof will already have been designed for additional snow.

Items such as parapet walls, fans and air handling equipment mounted on the roof should be noted. These items affect both the layout of the collectors and the clearance beneath the collectors. Collectors have to be located so as not to interfere with the operation of mechanical equipment. Reduction of the effective clearance between the underside of the collectors and the roof by such items may also reduce the scouring effect of the wind beneath the collectors and thus cause snow drifting.

3.02 Reinforced Concrete Structures

A preliminary site reconnaissance of a reinforced concrete structure normally reveals the general condition of the structure. It is also possible to determine basic beam and column dimensions at this time. Determination of slab depth is difficult but it may be measured where ducts or pipes pass through the slab or at stairwells. The more important aspects of the structure, such as the reinforcing steel, are, or should be, embedded in the concrete and cannot be seen. Some other items which should be noted during a site visit are:

(a) consolidation of the concrete - is a honeycombing significant?
(b) spalling of concrete - depth and scope;
(c) reinforcing steel cover - are bars exposed?
(d) other obvious distress - cracking, fracture, rusting of reinforcement, excessive deflection, etc.
(e) span to depth ratio - L/25 for beams
- L/30 for one-way slabs
- L/30 to L/33 for two-way slabs.

Composite Construction

Sometimes composite concrete/steel structures are encountered. This type of construction uses concrete one-way slabs spanning between steel beams. The steel beams act compositely with the concrete slab to increase the strength of the beam. It is important at this stage of the investigation to determine the means of shear transfer between the beam and the slab. If composite action depends on friction and only the top flange of the steel beam is embedded in the concrete, the beam will probably be overstressed under current Code requirements. If the beam is encased completely within a concrete beam, full composite construction is achieved which is recognized by our current Building Code. Current codes require the use of positive means of shear transfer between the steel and concrete, such as Nelson Studs or other shear lugs. This eliminates the possibility of abrupt failure of the compostie slab by failure of the inter-material shear face between the steel beam flange and the slab.

3.03 Steel Structures (beam/column/deck)

A good preliminary site reconnaissance of structural steel building provides the Engineer with a considerable amount of information on which to base a preliminary analysis.

Age

The approximate age of the building can often be determined by looking at the construction. Rivetted building structures were seldom constructed in Canada after 1955. After that date, steel structures tended to be connected by shop welds and field bolts. Built-up plate

girders, beams and columns were generally phased out of popular use about 1945 when larger and heavier rolled sections became available after World War II.

Measurements

Preliminary measurements should be taken to verify information shown on design drawings. Span lengths should be checked. Beam and column dimensions should be verified. All rolled sections should be classed as either wide flange sections or I sections (now called standard beams or S sections). Wide flange sections have flat (horizontal) inside flanges and relatively low web/flange thickness ratios whereas I beams have sloping (tapered) inside flanges and relatively high web/flange ratios.

Continuity

The structure should be examined for continuity. Are beams simple span, continuous or semi-continuous? Semi-continuous construction, shown in Figure 1, sometimes known as Gerber-type construction, uses cantilever beams over the columns and drop-in or link beams. It is 20% to 30% more efficient for bending moment than simple span-construction. Fully continuous frames should be identified and the moment-resisting connections should be examined to determine whether they are full moment connection or whether they transfer only a portion of the capacity of the sections involved.

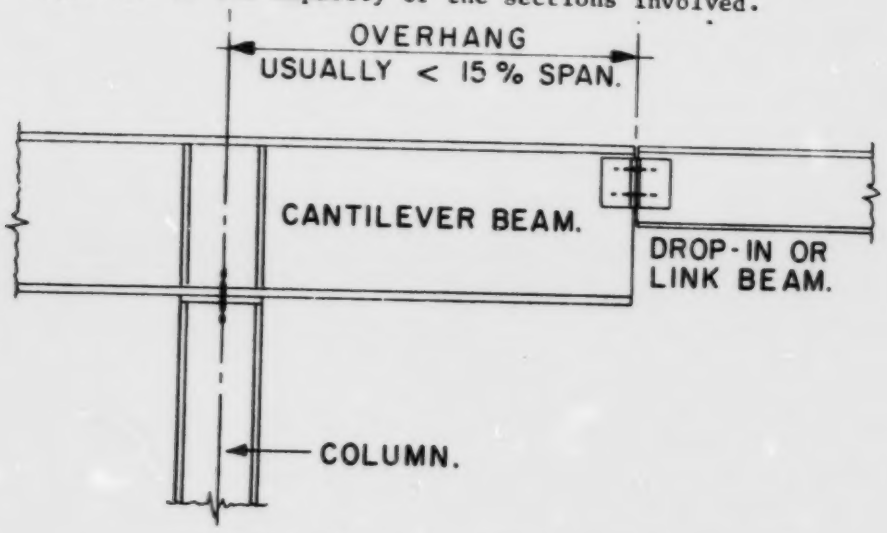

FIGURE -1 - Typical Gerber Construction

Condition

The condition of the structure should be examined. Has corrosion (rusting) occurred? If so, why? Rusting at connections is generally more serious than rusting in the span of the beams. Is distress or excessive deflection noticeable? Either could indicate a previous overload which may have seriously weakened the structure. The overload may have occurred as a result of normally applied loads, thus indicating a weak or inadequate structure. Or it may have been an unusual occurrence and does not necessarily indicate a poor structure.

Rules of Thumb

Common rules of thumb which are useful to keep in mind during a preliminary evaluation of a steel structure are:

(a) Depth of a beam in inches should be approximately one half the span of the beam in feet.

(b) Weight of a beam in pounds per foot sized as in (a) should be about equal to the uniform load on the beam in kips per foot multiplied by the span of the beam in feet.

(c) Span to depth ratios for structural members should generally be in the following ranges:
- joists L/20 to L/22
- beams L/22 to L/24
- girders L/12 to L/16
- primary trusses L/8 to L/12

Composite Construction

Steel beams acting compositely with a concrete deck are a common type of construction. The method of shear transfer between the beam and slab should be determined. Unless shear lugs are used or unless the steel beam is completely encased in reinforced concrete, composite action cannot be assumed. Beams with only their top flanges buried in the slab and without shear lugs to transfer the shear, cannot be considered to act compositely because of the abrupt failure exhibited by such beams at higher loads. Refer to Appendix 'A' for the dates before which the use of shear connectors became mandatory.

3.04 Steel Joist Structures

Light structures using open web steel joists as secondary framing members in a structural steel frame or in a bearing wall type buildings are very common. Such structures seldom have reserve structural capacity which can be tapped to permit increasing roof loadings. During a preliminary site reconnaissance, sufficient information about the construction should be obtained to permit a preliminary analysis to be carried out. Such information includes:

1) decking (depth and span);
2) geometry;
3) chords;
4) webs;
5) condition;
6) bridging and lateral restraint of panels;
7) joist tags.

Deck

The deck supporting the roofing should be examined to determine its approximate capacity, its ability to contribute to load sharing between joists and its ability to distribute loads to the panel points of the joists.

Geometry

Site information necessary to carry out a preliminary check of joist capacity includes determining the size of the top and bottom chords, the number of panels, the approximate configuration of the web (Pratt, Warren, etc.) and the size and location of the first and second web members from the end. With this information, a crude analysis can be rapidly made of the joist capacity.

Chords

Examine the chord members to see if they are hot rolled or cold rolled members. Hot rolled sections have sharp 90° corners. Cold rolled sections have rounded 90° corners. Cold rolled section have much higher yield strengths due to strain hardening of the steel during the cold rolling process.

The maximum panel length of the top chord should be verified. Combined compressive and axial load, rather than just axial load must be considered for compression chords in which panel spacing exceeds 610 mm.

Look for ceiling extensions of the bottom chord beyond the end diagonal web member. Ceiling extensions can be cut if billet-testing or coupon testing is required. No replacement of the ceiling extension is usually required. There is no axial load in the extension to account for when the specimen is cut.

Webs

Determine the type and configuration of the web members. Flats were often used as tension web members. Recent changes in building codes require unbalanced loading conditions and minimum point loads to be considered. This may cause stress reversals in some of the tension web members located near the centre of the joist. Flats have little or no compressive capacity to accommodate the stress reversal.

Condition

The joists' condition should be examined. Welds, especially of the end tension diagonals, should be checked. Excessive sag in the joist, twisting or other misalignment should be noted.

Bridging and Lateral Restraint

Look for the lateral restraint mechanism which allows the slender joists to work in-plane. Is the top chord of the joist properly braced? In buildings constructed prior to 1960, compression chord bracing was often accomplished by driving wood wedges through the metal pan formwork when thin concrete deck slabs were used. Otherwise the top chord should be embedded in the concrete deck or securely welded to the metal deck.

Top chord bridging is seldom spaced closely enough together to be considered adequate lateral restraint for the top compression chord of the truss. Top chord bridging is generally considered to be temporary or construction bracing for the joists until the permanent lateral support of the deck is connected.

Similarly, bottom chord bridging should be considered as a construction measure rather than a permanent type of bracing unless there is stress reversal causing bottom chord compression.

Joist Tags

Locating a joist tag may provide loading information for a joist. Major joist manufacturers such as Dominion Bridge, Massilon or Ingals, published tables for their joists for many years. These tables are available at most major engineering libraries.

4.00 PRELIMINARY ANALYSIS

4.01 Loads

Loads used for the preliminary analysis should not be conservative. They should be realistic! This applies particularly to the existing building structure dead loads.

Sufficient information should have been gathered during the preliminary site reconnaissance to allow a reasonable estimate of dead loads to be made. Mechanical lighting and ceiling loads should be assessed carefully. Unless specifically instructed by the Client, it should not be necessary to provide reserves in the dead loads for future alterations.

Dead Loads

Dead loads of the collector system, for the purposes of preliminary analysis may be assumed to be about:

.2 kPa for collectors - based on the size of the collector

.1 kPa for collector supports - based on the footprint of the collector

.05 kPa for piping, etc. - based on the footprint of the collector

For example, the estimated dead load of a 2-high collector system with 1.20 x 2.40 m panels and a 45° slope would have an estimated dead load as follows:

collectors	.20 kPa x $\frac{1}{\cos 45°}$	=	.28 kPa
supports		=	.10 kPa
piping		=	.05 kPa
			.43 kPa based on footprint area

Wind and Snow Loads

Wind and snow loads should be taken from PWC Briefing Document S2 Structural Design for Solar Collectors.

4.02 Material Properties

Material property assumed for the preliminary analysis should not be conservative. Rather, they should approach an upper bound.

Material properties specifications from existing drawings or specifications should be used if available. If drawings and specifications are not available, an estimate of the properties must be made. Appendix 'B' provides information on material properties for structural steel and for reinforcing steel but not for concrete strength. Determine the approximate date when the building was constructed. Then, refer to Appendix 'B'. Material properties based on the Codes in use at the time of the construction can be chosen.

Reinforced Concrete

The flexural capacity of reinforced concrete members (analysed by Ultimate Strength Design) is quite insensitive to concrete strength but is very sensitive to reinforcing steel strength.

For example, consider a 240 mm slab reinforced with 20 M bars @ 120 centres. The flexural capacity of the slab increases by only 3% to 4% when the concrete strength is increased from 20 MPa to 30 MPa (50% increase) and the reinforcing steel has a yield strength of 300 MPa. The flexural capacity of the slab increases about 30% when the concrete strength is held constant at 20 MPa but the yield strength of the steel is increased from 300 MPa to 400 MPa (33% increase).

The shear capacity of a concrete structure is sensitive to concrete strength in areas which rely on only the concrete to resist shearing forces (such as flat slabs). Structures which rely on a combination of reinforcing steel and concrete to resist shear and are not as sensitive to concrete material property assumptions (such as beam and slab type construction).

Structural Steel

Structural steel and steel joist structures have shear and flexural strengths directly proportional to the yield strength of the steel. At this stage of the investigation, the Code provison - that all unidentified steel be assumed to have a yield strength of 210 MPa - should be ignored. The assumed yield strengths for the preliminary analysis should be based on the type of steel can be available at the time. Later the strength properties of the steel can be verified if the structure shows promise of being able to accommodate additional loads.

4.03 Structural Analysis

Strength Design

The preliminary analysis should be carried out using Ultimate Strength Design or Limit States Design techniques, not Working Stress Design Techniques. With heavy structural systems such as reinforced concrete 2-way slabs, beams and slabs, or structural steel and concrete deck, the reserve capacity available using Ultimate Strength or Limit States considerations over Working Stress Design is very significant.

Good Practice

At this stage, the structure deserves the courtesy of a good analysis. Proper allocation of loads using code recommended tributary areas to structural members should be used in assigning loads to members. Avoid using generalized uniformly distributed loads for the analysis of the primary members of a structural steel frame. Instead use discrete point loads imposed on the beams or girders by secondary framing members such as purlins or joists.

5.0 GUIDELINES ON PROCEEDING

Having completed a preliminary evaluation of the structure the question arises as to whether further time, effort and money should be expended on engineering to carry out a detailed analysis of the structure.

If the structure is adequate by a large margin, the assumed or unverified material properties can be reduced to Code specified minimums for untested materials and the structures can be reanalysed.

If the structure appears to be marginally adequate, ie. no overstresses, on the basis of the preliminary analysis, proceed with the securing of material properties and the carry out the detailed analysis.

If the structure is overstressed under design loads by more than 10%, it is unlikely the structure will benefit from further testing and analysis. Therefore, it would be prudent to suspend further engineering work unless funds are available in the budget for strengthening the structure.

A grey area occurs in the decision process when the preliminary analysis yields strength results within 10% of the required capacity. If the structure is reinforced concrete, material testing and a detailed analysis are warranted. If the structure utilizes heavy steel construction, further material testing and detailed analysis is warranted. Light steel joist structures will not have the additional capacity so materials testing and conducting a detailed analysis would not be warranted.

6.0 REFERENCES

1. "Structural Design for Solar Collectors", Public Works Canada Briefing Document S2.

2. CAN3 A23.3-M77, "Code for Design of Concrete Structures for Building".

3. CSA S16.1-M78, "Steel Structures for Building - Limit States Design".

4. "Handbook of Steel Construction", 3rd edition, Canadian Institute of Steel Construction, 1980.

CHAPTER 4

EVALUATION OF EXISTING MATERIAL PROPERTIES

CHAPTER 4

EVALUATION OF EXISTING MATERIAL PROPERTIES

1.00 INTRODUCTION

Having completed a preliminary analysis of the structure and having determined that it is worthwhile carrying out a detailed analysis, how do you proceed? You need accurate physical information and verified material properties to carry out detailed analysis. This chapter deals with the variety of equipment available for determining physical information and verifying in-situ properties of structural materials.

The information necessary for a reliable analysis of a reinforced concrete structure is:

a) concrete strength (equivalent 28-day strength)
b) reinforcing steel strength (yield stress)
c) reinforcing steel size and location

The size and location of the reinforcing steel are geometric properties of the structure, not material properties. Determination of reinforcing steel size and location is included in this chapter as it relates well to the advanced site techniques and equipment used to obtain other material property information about the structure.

The key material property, which must be obtained to carry out a reliable analysis of a steel structure, is the yield strength of the steel.

2.00 CONCRETE STRUCTURES - IN SITU MATERIALS

The following destructive and non-destructive testing techniques can be used for verification of material properties and for locating reinforcing steel.

a) Concrete strength - Coring
- Surface Hardness - Windsor Probe and Schmidt Hammer
- Pull-out testing

b) Reinforcing steel strength - Coupon (billet) testing

c) Reinforcing steel size and location - Drilling and chipping
- Magnetic (Pachometer)
- Radiography (x-ray)

2.01 Concrete Coring

Description

The cutting and testing of concrete cores to establish the compressive strength of in-situ concrete is a common, well documented and well recognized technique. It involves cutting of concrete cores from the structure using a thin wall diamond core drilling machine, trimming the cores with a diamond saw, and testing the cores for compressive strength utilizing traditional capping and cylinder testing techniques.

Destructive

The procedure is destructive. Holes are left in the structure which must be subsequently filled, but it is seldom destructive to the point that shoring of the structure is required or that the cored structural member has to be repaired structurally. Water is used as a coolant and a lubricant in the cutting of the core and the procedure is messy. Using a wet vacuum in conjunction with the coring reduces the mess but invariably water is spilled and has to be dealt with.

Code Provisions

The procedure for obtaining and testing concrete cores is described in CSA Standard CAN3-A23.2-M78 ([16]), Sections 9C and 14C.

Precautions

Core test results can be significantly influenced by utilizing certain techniques or by omitting certain practices. It is important, therefore, to recognize what influences core strengths before proceeding with a testing program.

In general, core strength is reduced by,

a) damage to cores during drilling

b) poor in-situ compaction of the concrete at core location

c) water gain in core from drilling

d) small core diameter.

Core strength is generally increased by,

a) age of the core versus 28-day control

b) loss of air entrainment in-situ

c) dry testing.

Cores should be cut such that a final cut-off L/D ratio of 2.0 is maintained where possible. The minimum L/D ratio of a core is 1.00 and results for cores tested with L/D ratios less than 2.0 should be corrected to 2.0 in accordance with the Code Provisions. The core diameter should never be less than 100 mm or less than three times the size of the maximum coarse aggregate. Small size cores tend to give unrealistic strength results.

Cores should be tested under conditions similar to their native environment. Generally, concrete taken from buildings exists in a dry condition. Therefore, the cores should be dried at least 24 hours at room temperature and humidity to eliminate the saturation caused by the drilling water.

Reinforcing bars should never be left in cores when they are tested. The rebar can be avoided when cutting cores if their locations are pre-determined by the use of a Pachometer (section 2.05). Otherwise, the reinforcing steel must be cut out of the core length to be tested.

Availability and Cost

Concrete core testing is a common type of testing operation. Most cities have concrete drilling contractors with equipment to cut the cores. Most geotechnical and testing engineering firms have the compression testing equipment to test concrete cylinders and this equipment can be used to test the cores. Most universities and community colleges offering engineering or technology courses also have concrete compressive testing equipment. Coring is relativing inexpensive. Generally, a set of three cores can be cut, capped and tested for about $200.00 (1983$).

Number of Tests

"How many core tests should be performed?" is a common question which comes up when dealing with the evaluation of structures. The ACI ([4]) recommend a minimum of two cores for single elements spanning less than 7.6 metres or areas less than 58 m^2 being investigated. For larger spans or areas, more tests are required. The Ontario Highway Bridge Design Code ([17]) requires a minimum of two cores per span, or in the case of a single span bridge a minimum of four cores. These are the only specific recommendations made in any code for the evaluation of existing structures.

Realistically, sufficient cores must be taken to obtain a reliable indication of both the average strength of the concrete and its variability. A minimum of three cores should be obtained from the slab portion of a flat slab type structure. For beam and slab structures, two cores from the slabs and two cores from the beams should be tested. Separate cores should be taken from columns if required for column analysis, as columns are often poured from different strength concrete than the beams and slabs.

2.02 Concrete Surface Hardness - Windsor Probe and Schmidt Hammer

There are two testing methods used to estimate in-situ concrete strengths which rely on a relationship between surface hardness and compressive strength. These are the Windsor Probe and the Schmidt Hammer.

The correlation between the instrument readings and concrete compressive strength in both methods is not accurate enough to provide data for a detailed analysis but the methods are in such common use that they are described herein for the benefit of the reader. They can often be used to supplement other testing techniques to increase the other techniques' accuracy.

Windsor Probe

Description

The Windsor probe is an instrument which fires a sharp hardened alloy probe into the concrete surface by a powder activated gun or driver. Because of this, it is sometimes referred to as a penetration method rather than a surface hardness method.

The length of the probe which remains projecting from the concrete surface after the probe is fired is measured. Each test consists of three probes, the results of which are averaged. The test, or average projection is then related to a calibrated scale and the compressive strength is deduced.

Destructive

The Windsor Probe test is non-destructive but it does cause cosmetic damage to the concrete. Such damage is easy to repair.

Code Provisions

This testing method is not regulated by any recognized building code standard. It relies on the manufacturer's written instructions to govern the testing techniques uniformity of application and interpretation of results.

Precautions

Calibration tables prepared by the manufacturer are included with the equipment. There are different calibration tables for concrete with varying hardnesses of aggregate. Depending on the hardness of the aggregate employed in the concrete mix, different compressive strengths result from use of calibration tables.

According to studies published in the ACI ([8]), the correlation tables published by the manufacturers are not very accurate for concrete except for the specific material for which they were calibrated.

Correlation tables for a specific type of concrete in use in the area can be prepared to increase the reliability of the instrument, but even with careful calibration of the probe for a specific mix, the results are not very accurate. The standard deviation and the coefficient of variation of the results are about 16 times greater than those for either cylinder or core strengths from the same concrete.

The following statements best describe the reliability and usefulness of the Windsor Probe in evaluating the strength of unknown concrete structures:

> "Because of the large variability in the probe test results, the usefulness of this approach lies in determining the relative quality of concrete in place rather than in its use as a means of quantitatively predicting the 28-day compressive strength of concrete." ([8])

> "Based on these tests, the probe system does not supply the accuracy required if it is to replace conventional core tests." ([8])

Availability and Cost

The Windsor probe is not a particularly common testing instrument. Few testing companies own such equipment and fewer will employ the technique for direct evaluation of concrete strength. The cost of a single test is approximately $35.00 (1983$) but for a series of tests, the unit cost reduces significantly because of the ease of use.

Use

The Windsor probe is not accurate enough to determine absolute values of strength in concrete. It should therefore, not be used to evaulate existing material properties in structures for analytical purposes unless it is used to augment or to confirm concrete strength results determined by other means. It can be used as a means of determining

relative strengths of concrete in differing parts of the same structure or relative strengths of different structures.

Schmidt Hammer

Description

The Schmidt hammer is a testing instrument which relates surface hardness of the concrete to compressive strength. A spring loaded hammer is released against a concrete surface and the distance the hammer rebounds is measured. Usually 15 such rebound readings are taken for a single test. The high and low results are discarded. The remaining readings are averaged and related to a calibration scale.

Precautions

A manufacturer's calibration chart relating rebound number to compressive strength is provided with the hammer but it can only be used when the material being tested and testing conditions are similar to those used for the original calibration. The hammer may also be calibrated for local concrete. It is suggested ([8]) that the accuracy of the hammer in terms of concrete strength is about 25% for concrete strength in a structure. Part of the variability is made up from the variation in surface hardness in the concrete and part is inherent in the correlation between surface hardness and compressive strength.

Destruction

The Schmidt hammer test is non-destructive.

Code Provisions

This test is not covered by any recognized building code standard. It relies on the manufacturer's written instructions to govern the testing technique's uniformity of application and interpretation of results.

Availability and Cost

The Schmidt hammer is a common testing instrument; almost all testing firms own one. The cost for use of a Schmidt hammer is usually added to the fee of the technologist or engineer who operates the hammer. Because of its simplicity and the speed with which a series of 15

rebound numbers can be taken, the cost of using a Schmidt hammer is very low.

Use

The Schmidt hammer should not be used to determine absolute concrete strengths, because of its high degree of inaccuracy. Rather, it should be used to determine relative strengths of concrete in differing parts of a structure in conjunction with a more precise and more accurate strength control measurement, such as concrete coring or concrete pullout testing.

2.03 Concrete Pullout Testing

Description

A recent development in the testing of concrete is the pullout test. It provides a rapid, accurate method of determining the in-situ compressive strength of concrete.

A calibrated tension ram is used to pull out a device embedded in the concrete. The test measures the combined shear and compressive strength of the concrete. This correlates very well to the compressive strength of the concrete.

The primary way in which such testing has been used to date is in planned testing programs. Inserts are cast into fresh concrete and after the concrete has cured, the inserts are pulled out. A less common approach, and one not nearly as well documented, is one in which inserts are put into holes which have been drilled into already hardened concrete. The inserts are secured in the holes by either epoxy glue or by use of an expanding sleeve. When the inserts are secure, they are pulled out and the pull out force is related to the concrete's in-situ strength.

Destruction

The hole left by removal of a pullout insert is conical. It has a maximum diameter of about 125 mm and a depth of about 50 mm. From the strictest point of view, the test is destructive, however, the

structural member tested need not be discarded and the hole can be readily patched. From a practical point of view, the test is dry and relatively clean to carry out - unlike core drilling.

Codes

The ASTM Committee on Non-Destructive Methods of Testing Concrete have issued a tentative standard ASTM C900-78T to standardize the subject. At present, the Canadian Standards Association have not developed or adopted a standard for pullout testing but it is understood to be in the planning stages.

Accuracy

Good correlation exists between the pullout and the compression tests. Malhotra ([6]) noted correlation co-efficients of over 80% both for epoxy grouted threaded inserts and for split sleeve inserts relating to drilled cores. Correlation co-efficients over 85% were obtained for the epoxy grouted threaded inserts and split sleeve inserts relating to cast cylinders. The relationship between cores and cylinders was found to yield a correlation co-efficient of about 90%. It has therfore been concluded that pullout tests are nearly as accurate as for drilled cores or for cast cylinders.

Number of Tests

The number of tests to evaluate the strength of concrete in the structure should be sufficient to obtain a measure of both the average strength and the deviation of test results from the mean. A similar number of tests to those taken for a coring program should be adequate (Section 2.01). A good rule of practice would be to average three pullout readings for each test required.

Availability and Cost

Pullout testing has only been recently developed on a commercial scale. Equipment is generally only available from large testing firms, especially those involved in multi-storey reinforced concrete buildings, as the method has particular relevance to the stripping of falsework. The cost, where available, is about $200.00 (1983$) for a set of six readings. This would yield two test results.

2.04 Rebar Coupon Testing

Description

The most common method of determining the in-situ yield stress of reinforcing steel bars involves removal of pieces of the reinforcing steel bar and carrying out a tensile test on that coupon.

Destruction

This method is destructive. However, in many cases the bars can be removed from an area of low flexural stress and hence the structural capacity of the member or slab from which it is removed is not compromised.

Codes

The testing of billet steel reinforcing bars is covered, in general terms, in CSA Standard G30.12-M1977, Billet Steel Bars for Concrete Reinforcement ([13]) and specifically in ASTM Standard A370, Mechanical Testing of Steel Products ([18]).

Precautions

In removing a specimen of reinforcing steel from the concrete, certain procedures should be observed to avoid damage to the coupon:

a) Remove a piece sufficiently long that tensile testing is possible. The coupon should be at least 400 mm long to permit the installation of a 150 mm extensometer for the testing.

b) Do not bend or kink the rebar sample.

c) Do not apply heat (cutting torch) to the coupon within 150 mm of any part of the coupon to be tested.

d) Test representative size bars! If the main flexural steel in a beam is 30 mm in diameter, do not test 10 mm bars. In general, it has been found that small reinforcing bars tend to have significantly higher yield stresses when tested than do large bars.

Number of Tests

Buildings are usually constructed of only one type of reinforcing steel, with the possible exceptions that:

a) columns of some buildings may have higher strength steels.

b) stirrups in beams and ties in columns may have been constructed of lower strength steels.

In this manual - flexural members (beams and slabs) are probably the elements of the structure being analyzed. At least three coupons should be cut from the elements to obtain an average rebar strength. Rebar strengths from the same size of bar should not vary by more than 15%. Otherwise, the extreme samples should be rejected and additional tests should be taken. Bars with different deformations have likely been manufactured in different mills and should not be considered to be of the same grade of steel. Separate tests should be made for bars from different mills.

Cost and Availability

Coupon testing is a well documented and common procedure. Chipping out the concrete, cutting the coupon, preparing and testing the coupon and reporting the results costs in the order of $125.00 (1983$) for three specimens.

2.05 Magnetic Testing (Pachometer)

Description

The Pachometer or cover meter is a portable battery operated instrument which can detect the size or the position or the cover of reinforcing bars in concrete.

The Pachometer consists of a probe and a separate box housing the power source, an amplifier circuit and a moving coil meter. The probe contains a highly permeable U-shaped magnetic core on which two coils are mounted. The unit works by creating a magnetic flux in one coil of the probe and measuring the induced current in the other coil. The

induced current is a function of either the nearness or the size of reinforcing steel bars in the magnetic flux linkage path between the two coils.

To determine concrete cover with a Pachometer, the size of the reinforcing bars being investigated must be known. Conversely, to determine the bar size it is necessary to know the concrete cover. The Pachometer may also be used simply to locate reinforcing steel if neither the size nor the cover is known.

Accuracy

The Pachometer gives satisfactory results when used on reinforced concrete members which are lightly reinforced and in which the reinforcement is located relatively close to the surface (less than 50 mm). Investigation of slabs and walls with the Pachometer generally yields good results. The accuracy of the cover readings are $\pm$6 mm ([8]).

In heavily reinforced sections, the effect of secondary reinforcement or closely placed reinforcement clouds the readings, making them inaccurate. In theory, the Pachometer will not differentiate between bars if the spacing is less than three times the concrete cover. Practically speaking, it is difficult to obtain accurate results when spacings are this close. The effect of perpendicular bars, such as stirrups or ties, makes determination of the size or cover of steel in beams and columns very difficult.

The use of the Pachometer, in conjunction with drilling and chipping, is probably the best method of defining reinforcing steel placement and size. The Pachometer is used strictly to locate bars - which it does very well - and then the concrete is chipped away locally to expose the bar or the concrete is drilled with carbine tipped masonry drill, to determine the size and cover. Using this combined technique, more heavily reinforced sections can be investigated quite accurately. The damage to the concrete is quite minimal; it is readily repaired. The structural integrity of the member is not reduced.

Limitation

The Pachometer is limited in its working environment due to its dependence on dry cell batteries as a power source. The dry cells do not produce adequate power to operate the Pachometer when temperatures are below 5°C.

Availability and Cost

A Pachometer costs about $2,000.00 (1983$) to buy. Most larger testing companies and some general consulting engineering firms own one. The rental rate is usually added on to the operating technician's hourly rate. The equipment rental rate alone is about $20.00 per hour. In budgeting for the use of a Pachometer, consideration should be given to the accessibility of the location where the testing is to be done. For example, if the positive flexural steel in a slab is to be investigated, the operator must work either on a ladder or on a scaffold. If the negative flexural steel in a slab is being investigated, the operator can work on foot.

2.06 Radiography

Description

Recently, x-ray technology has become available on a relatively portable basis. It is now being used to locate and size the reinforcing steel in reinforced concrete structures.

A gamma radiation source (which weighs about 225 kg) is located on one side of a structural element and placing an x-ray sensitive photographic paper is placed on the other size of the structural element. The photographic paper is exposed to the radiation and an x-ray image of a negative form is created. By knowing the thickness of the structural element and the distance of the radioactive source from the photographic paper, the size and location of the reinforcing steel can be determined. However, the reduction of the data to a useable form involves mathematical calculations and drafting of the results.

Destruction

The operation is non-destructive but the precautions necessary to shield the operators and other occupants of the building may cause more inconvenience than some destructive testing techniques.

Limitations

The technique is expensive and time consuming. In some cases it may be the only way in which information can be obtained non-destructively. It is not commonly available due to its high capital cost, due to the high level of skill required of the operating technicians and due to expense of testing.

2.07 Other Techniques

Several other concrete testing techniques are in common use to determine in-situ properties of concrete but their accuracy for use in determining absolute material properties is questionable. Manufacturers of such equipment are often prone to exagerate the abilities of their product to perform a secondary function for which it is not really suitable or accurate. Correlation of instrument readings to concrete compressive strength is a common claim made by many manufacturers. The structural engineer investigating the existing condition of a building is often bombarded by claims of reliability for particular instruments in lieu of coring or pullout testing, especially if the equipment is readily available or inexpensive.

The Schmidt Hammer and Windsor Probe really fall into this categoy of instruments with unreliable correlation to compressive strength. Other instruments in this category, which are not sufficiently accurate for determination of absolute concrete compressive strength are:

1) Ultrasonic testing equipment (Accoustic emmission)
2) Surface hardness methods
3) Penetration techniques
4) Nuclear methods

5) Electrical methods

6) Microwave absorption techniques

7) Dynamic or Vibration methods

All such equipment should be avoided as a primary source of determining absolute compressive strength. Some are, however, useful as secondary checks as to the uniformity of concrete strengths in the structure.

3.00 TECHNIQUES FOR STEEL STRUCTURES

3.01 Structural Steel Coupon Testing

Description

The cutting and testing of steel coupons from in-situ structural steel is a common, well documented and well recognized technique for establishing the yield strength of the structural steel. A coupon of steel is cut from an area of low stress in the structure, machined and tested in a tensile testing machine.

Destruction

The method is destructive but if the coupon is cut from an area of low stress in the structure, temporary shoring is seldom required and repairs may not be needed. In some cases, this is not possible and shoring and repairs do have to be carried out.

Codes

The details for machining the coupon are covered in CSA Standard G40.20 M77 ([14]), General Requirements for Rolled or Welded Structural Quality Steel. The procedure for testing the coupons is specified in ASTM Standard A370, Mechanical Testing of Steel Products ([18]).

Precautions

Coupons removed from the steel structure should initially be cut out as rectangular pieces about 100 mm wide x 500 mm long. The coupons may be either flame cut or saw cut from the member. Machining for

testing will remove any metal affected by the cutting operation. Under certain circumstances, such as testing joist chords or webs, smaller coupons may be acceptable. This should be verified with the agency carrying out the testing.

The coupons should be cut from areas of low stress. The following locations are recommended for structural members:

a) Beams - centre of webs at approximate 1/3 point of span - simple or continuous beams
   - bottom flange at support - simple span beam.
b) Columns - centre of web.
c) Joists - ceiling extension of bottom chord.
d) Trusses - extension of bottom chord to column or support.

Representative steel coupons should be taken from the structure. For example, coupons should not be cut from a column web if the governing stress condition is the bottom (tension) chord of the joist or the tension flange of the beam. Coupons should be cut from similar low stress areas of a member to that area which will govern the stress.

In some cases, it may be impossible to remove a coupon without first shoring up the structural members and immediately welding a replacement section of steel into place. In welding material back into the member, care should be taken to avoid creating stress concentrations caused by tack welding, or by welding across tension flanges of beams or by welding across tension chord members of trusses or joists unless it is certain that fatigue will not occur at the weld.

Accuracy

The coupon test is extremely accurate. It correlates directly to the yield and ultimate strengths of the member tested. The location in the parent member from which the coupon is cut has a significant bearing on the coupon test results. Coupons cut from the web of a wide flange member will produce higher yield strengths than those obtained from the flange. This is because the webs are rolled

considerably thinner than the flanges and are thus subject to more work hardening or stress hardening, thus elevating the yield stress. In general, it can be assumed that the difference in yield stress between the web and flange of a wide flange beam varies from 4% to 7%.

Cost and Availability

Coupon testing is routinely carried out on steel for buildings and bridges. Most universities and some larger testing companies have both the machining equipment and the tensile testing machine to carry out the work. The cost of a coupon test is about $100.00 to $150.00 per coupon (1983$), including the cost of cutting the coupon. The costs can run significantly higher if a shoring and/or repairs to the structure are required.

Number of Tests

How many tests should be done? The Ontario Highway Bridge Design Code ([17]) recommends one test be taken for every 50 tonnes of steel in a bridge. However, the uniformity of steel specified in a bridge is usually more constant that in a building. In a building many components of the structure may be fabricated from different types of steel. For example, a warehouse building might have HSS columns with a nominal yield stress of 350 MPa, wide flange beams with a nominal yield stress of 300 MPa, joists with cold rolled chords having a yield stress of 375 MPa and webs with a yield stress of 300 MPa.

The preliminary analysis should indicate which members are likely to be stressed to a critical degree and coupon tests should be carried out on those members. In the above example, the joist chords, the joist webs and the beams would probably be critical and require testing.

As a general rule, a minimum of three coupons should be taken for each type of steel being investigated. The coupons should be taken from a consistent location, for example, the beam web, and the results should be averaged. Test results which differ from the average by more than 15% should be disregarded and additional coupons obtained.

3.02 Surface Hardness

Description

A technique that has been around for a long time but which has only recently been correlated to structural steel strength testing, is that of hardness testing ([11]).

Basically, the technique involves the use of a portable Rockwell hardness testing device to establish the surface hardness of the structural steel. A good correlation exists between the surface hardness and the yield strength. Recent tests indicate a correlation co-efficient in excess of 80%.

Destruction

The technique is non-destructive and can therefore be used in-situ without incurring the costs of shoring, cutting and repairing the structure.

Codes

Hardness testing is not yet covered in any of the building codes as a recognized technique for determination of yield or ultimate strengths in steel.

Procedure

To obtain hardness readings the structural steel is first cleaned down to bare metal, removing all rust and mill scale, with either emery paper or a portable grinding wheel. The hardness tester is then clamped to the member and three hardness readings are taken using three different test loadings. The three different readings are known as Rockwell F, Rockwell B and Rockwell G scales and use loads of 60, 100 and 150 kg respectively. The tester is then moved and similar triple readings are taken at three or four other locations. The readings for each scale are averaged and then plotted on a calibration graph. The best fit of a straight line through the three points is then extended to intersect a portion of the graph indicating yield strength.

Limitations

There are limitations on the use of the method due to the extremely limited data used to establish the correlation. It should not be used on any steel which has been hardened, either by cold working or by heating. This means the method cannot be used on cold formed sections such as joist chords or on any areas of structural steel which have been locally influenced by heating such as by flame cutting or by welding. The method is not effected by residual stresses (meaning all dead and live load stresses as well as manufacturing stresses) as long as the total residual stress is below the yield stress. Therefore, testing locations can be carried out at whatever location is considered to be critical to the structure.

Number of Tests

At least three tests for each type of steel being investigated. Each single test should include three readings (F, B and G scales) at three or four locations.

Cost and Availability

An individual set of readings can be carried out in about five minutes while a complete set of three to four sets can be completed in about 15 minutes. A portable Rockwell hardness tester costs between $1,200.00 and $1,800.00 (1983$) and is all the equipment really necessary for the testing other than emery paper or a portable drill with a grinding wheel. These testers are not common in testing firms. Many universities have full size testing equipment but some have portable units for students' lab use.

4.00 INTERPRETATION OF DATA

It is a common misconception that in-situ testing strength results can be used directly in a structural analysis without interpretation or modification. This is not so! The material properties must, in most cases, be modified to conform to the intent of the building code.

The current building codes pertain specifically to new construction in which new materials are specified. The materials in the Codes are governed by specifications detailing rigid quality control. In evaluating an existing structure, the formulation of the code clauses must be taken into account insofar as the material properties are concerned. Material properties are generally specified in codes so there is a 90% probability that the greater strengths will be achieved. Using an average strength from testing results means that only 50% of the time greater strengths will be achieved. Accordingly, modification of the test results is necessary to move from the 50th to the 90th percentile of confidence.

4.01 Concrete

The in-situ strength of cast-in-place concrete (fc) is related to the cylinder strength (fc') The cylinder strength (fc') is used for the analysis or the design of new structures and forms the basis of the concrete building code. There is both a statistical and a physical difference between these two values. The relationship of these two seemingly very similar properties is briefly examined herein.

Fc' value, a 28-day lab-cured test cylinder strength, is a minimum strength specified such that there is a probability of not more than

a) 1/100 that three consecutive tests will be below fc'.
b) 1/100 that any one test will be below fc' by more than 3.5 MPa.
c) 1/10 that a single random test will fall below fc'.

Therefore, the concrete supplied to a project must have an <u>average</u> concrete strength substantially higher than fc'. For good quality concrete (standard deviation less than 3.5 MPa) the average strength would be about 5 MPa greater than fc'. For poorer quality (ie. less than quality control) the average strength may be up to 8.4 MPa greater than fc'.

Core sampling of in-situ concrete yields average strength results (fc), not minimum strength results. If the design strength of a

member was to be related to an average concrete strength, such as the average of the core results, then 50% of the time below strength results would occur. Therefore, the in-situ strength (fc) must be modified before using it in building code design equations.

Besides a statistical difference between in-situ concrete strength (fc) and the design strength (fc'), there are other significant factors which cause a difference between these values. Cylinder tests for concrete provide a guideline as to the expected strength of a particular mixture of concrete which is cured under controlled conditions. Field curing conditions often bear little or no resemblance to the lab curing conditions. Field test results therefore often differ considerably from the lab cured cylinder tests. A list of the key factors which affect the strength of in-situ concrete include:

1. age
2. consolidation
3. temperature
4. surrounding moisture

Age

Generally, when investigating an existing structure for solar collectors which requires material testing of the concrete, the structure is significantly older than the 28-day age used for the cylinder test. Otherwise, structural drawings, specifications and material properties would be available and field testing would be unnecessary. Age affects the strength of concrete. Hydration of the cement continues almost forever unless the concrete is artificially dried. In some types of concrete, the actual strength achieved is up to twice the specified 28-day strength. More realistically though, strength increases of 10% to 20% beyond the 28-day strength of the concrete are common after two to five years. The design equations in the concrete building code relate to a strength of 28 days rather than a five year or greater concrete strength. This extra strength gain may be considered a bonus for analysing an older building. It is a recognized phenomenon but it is not included in the deviation of the performance factors used in the codes ([5]).

Consolidation

The consolidation of in-situ concrete can vary considerably. The effect of this variable on core testing can be minimized to a great extent by choosing a test location which appears from the surface to be well consolidated, then after the core is tested, confirming the degree of compaction. However, even the best consolidated field concrete is seldom as dense as an equivalent test lab cylinder.

Curing

The curing temperature and moisture conditions surrounding concrete greatly affect the hydration of the cement and thus the end strength. In-situ concrete tends to be cured at varying temperatures and generally in low humidity conditions. These conditions tend to reduce the hydration of the concrete and thus result in somewhat lower field strengths when compared to lab-cured cylinders. This is reflected in CAN3 A23.1-M77 ([16]) requirements as well as ACI 318-77 ([19]). Cores are required only to achieve an average strength such that fc $\geq$ .85 fc' (average) with no single test fc $<$ .75 fc' if low strength cylinder results are obtained.

Summary

McGregor ([5]) (1976) suggests that many of these factors counterbalance each other to some degree and that the "mean 28-day strength of concrete in a structure with minimum acceptable curing can be taken as:

fc^{28} = (.675 fc' + 7.6) MPa but not more than 1.15 fc'." ([1])

If we accept that any in-situ strength gain after 28 days is a bonus, then,

fc' = 1.48 fc - 11.3 MPa but not less that .87 fc ([2])

Thus if the average strength tests from a slab in a building are say 20 MPa, the design value for use in the analysis should be 18.3 MPa. Table 1 shows measured field strength versus design strength.

In-Situ Strength Average	Design Strength
fc (MPa)	fc' (MPa)
20	18.3
22	21.3
24	24.2
26	27.2
28	30.1
30	33.1
32	36.1
34	39.0
36	42.0
38	45.0
40	47.9

TABLE 1

4.02 Reinforcing Steel

The variability of strength in reinforcing steel is not a function of the field conditions. It is a function of the manufacturing process from which the bar is produced. Some of the factors in the manufacturing process which affect the strength variability are:

a) bar size (cross-heat variability)
b) in-batch variability
c) grade manufacture.

The strength of reinforcing steel found in the structure varies significantly if different grades were specified for different structural elements.

During the manufacturing process, all heats of steel are subject to mill tests to ensure they conform to a grade. Any steel not meeting the specified yield strength is rejected. Therefore, on site, there should be no steel specimens below the design minimum yield strength.

When coupons of reinforcing steel are cut from the structure and tested, a very small sampling is usually taken because of cost and disturbance. The sampling represents a very crude average steel strength. The building codes on which the design is based on are using a minimum steel yield strength, not an average steel yield strength.

Specified yield strengths in the codes have traditionally been separated into grades which have discrete, widely separated values, such as 276 MPa, 345, MPa and 413 MPa. There is up to 25% difference in minimum strength between grade. The approach of the ACI subcommittee on Strength Evaluation of Existing Concrete Buildings ([4]) therefore, is recommended for interpretation of coupon results as relating to a design steel yield strength. Table 2 is a summary of the recommendations.

Assumed Yield Point or Yield Strength of Reinforcing Steel in Place to be used for Analysis				
fy values from tests falling within intervals	227 MPa up to but not including 276 MPa	276 MPa up to but not including 345 MPa	347 MPa up to but not including 413 MPa	413 MPa and greater
fy values to be used for analysis	227 MPa	276 MPa	347 MPa	413 MPa

TABLE 2 ([4])

4.03 Structural Steel

Like reinforcing steel, the variability in the yield strength of structural steel is generally independent of the field conditions under which it is installed in normal buildings. The variability depends on manufacturing process and in particular is subject to change due to:

a) Rolling conditions

b) different heats of steel

c) in-batch variability.

Specifications for structural steel are based on a minimum permissable yield stress which is defined in such a way that the understrength steel is almost a 0 probability. All heats of steel are tested at the mill and understrength steel is rejected. Thus, the average strengths of steel in the structure can be expected to test considerably higher than the minimum specified yield strength.

The discrete grade into which hot rolled structural steel has been traditionally produced are much more tightly spaced in terms of yield strength than for reinforcing steel. The common grades have been 207 MPa, 227 MPa, 248 MPa, 262 MPa, 276 MPa, 303 MPa, 345 MPa. To assign a design yield stress grade to be equal to the minimum coupon test might be unduly conservative. To assign a design yield stress to be equal to the average of the coupon tests might be unsafe. Instead, take 90% of the average yield strength of the coupons or other tests for the design yield strength. This agrees closely with the results of the slightly skewed normal distribution for the strengths of structural steel

$$f_{y\,design} = .9\, f_{y\,average\ in\text{-}situ} \qquad (3)$$

5.00 REFERENCES

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CHAPTER 5

DETAILED ANALYSIS OF REINFORCED CONCRETE STRUCTURES

CHAPTER 5

DETAILED ANALYSIS OF REINFORCED CONCRETE STRUCTURES

1.00 INTRODUCTION

This chapter deals only with the analysis of reinforced concrete horizontal load carrying members, such as slabs and beams. Vertical load force carrying members, such as columns, are not treated herein because they are seldom found to be critical elements in the analysis of structures for the installation of solar collectors.

During the evolution of reinforced concrete construction there has been a great variety of changes both in materials used and in design. Higher strength materials have come into common use due to improvements in quality control in the concrete manufacturing business and due to new technological developments in the steel making industry. The codes for the design of reinforced concrete have similarly evolved to accommodate the changes in materials and to reflect the growing sophistication of building technology. Recently, a major change in the design philosophy, away from "working stress design" method to the "ultimate strength design" or "load factor" design method has occurred. There has also been a trend to move away from empirical formulas towards more sophisticated analysis methods such as elastic and plastic analysis, which model the actual stress conditions better.

Inevitably, as construction, design and analysis techniques have become more sophisticated, redundancies have been removed from buildings and overall safety factors been reduced. To ensure that safety factors will still remain at an acceptable level, additional clauses have been added to the codes. Bracing requirements have become more precise. Clauses to protect structures against abrupt or premature failure have had to be added. Compatability between various elements of structure must now be recognized in the analysis and serviceability criteria have been introduced to ensure that structures are functional and habitable as well as safe.

2.00 ULTIMATE STRENGTH DESIGN

Use

The most significant recent change in the design of reinforced concrete structures, has been the inception of the "ultimate strength design" or "load factor design" method. The use of this analytical technique alone, when compared to an analysis by "working stress" or "allowable stress" methods, can result in increases in the safe live load capacity of up to 25%.

The Ultimate Strength Design method must now be used for the design of all new construction in reinforced concrete. CSA A23.3-M77([1]) no longer recognizes the "working stress" method as valid because analysis by the "working stress" methods may result in seriously underestimating conditions involving load reversal.

For the analysis of existing buildings, ultimate strength design should always be used!

Original Load Factors

When Ultimate Strength Design was first introduced in Canada into the design codes (1965 National Building Code) load factors were higher than at present. The original factors were as follows:

1.5 for dead loads
1.8 for live loads, earth pressures

Current Load Factors

As the technique became more refined, the load factors were reduced, reaching their present values in the 1973 CSA Standard A23.3 and in the 1975 National Building Code. Performance factors were also modified.

Current load factors for reinforced concrete applicable to building construction are:

1.4 for dead loads, temperature effects and hydrostatic pressure
1.7 for live loads and wind loads
1.7 for earth pressures
1.8 for earthquake loads

In combinations where two or more of these loads act simultaneously on a structure, their overall effect may sometimes be reduced by a load combination factor.

Future Load Factors

These load factors are expected to be significantly reduced in the near future to bring them in line with current limit states design load factors (see Section 6 on Steel). Correspondingly, the performance factor factors for concrete will probably be increased to maintain adequate factors of safety.

3.00 ONE WAY SLAB AND BEAM

This section includes standard concrete floor joist construction as well as slab and beam construction. An example of a typical one-way slab on beams analysed by states design is included in section 3.06.

3.01 Loads

It is presumed at this stage of the analysis that all loads have been carefully examined to reduce them to the minimum acceptable level. Two types of loads must be considered, uniform loads and point loads.

Uniform Loads

Live Load Reduction

No live load reduction factors should be applied to uniformly distributed snow loads for slabs or beams.

Tributary Area

The tributary area for a slab supporting a uniformly distributed load is defined as: "The area bounded by the lines of the support"([3]). The tributary area for beams and girders supporting slabs carrying a uniformly distributed load is defined as: "The area enclosing the member and bounded by the lines of zero shear in the members supported"([3]). Such areas are shown in figures 1 and 2 reproduced from Commentary G of the Supplement to the 1980 National Building Code.

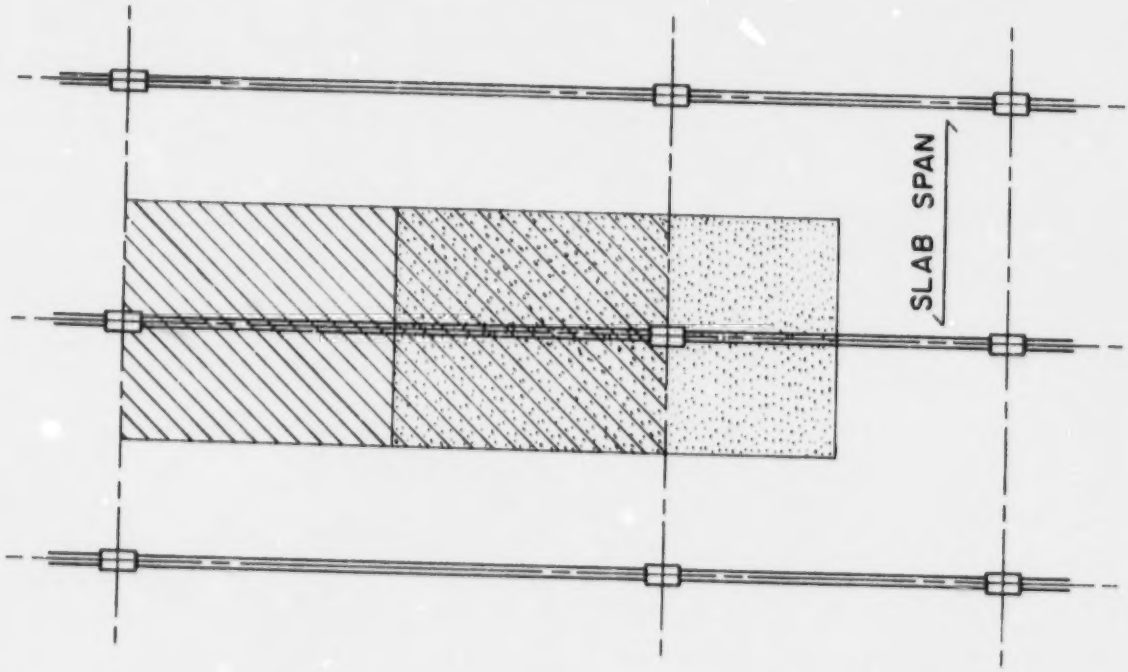


FIGURE 1 - Tributary Areas for One-Way Slab and Beams

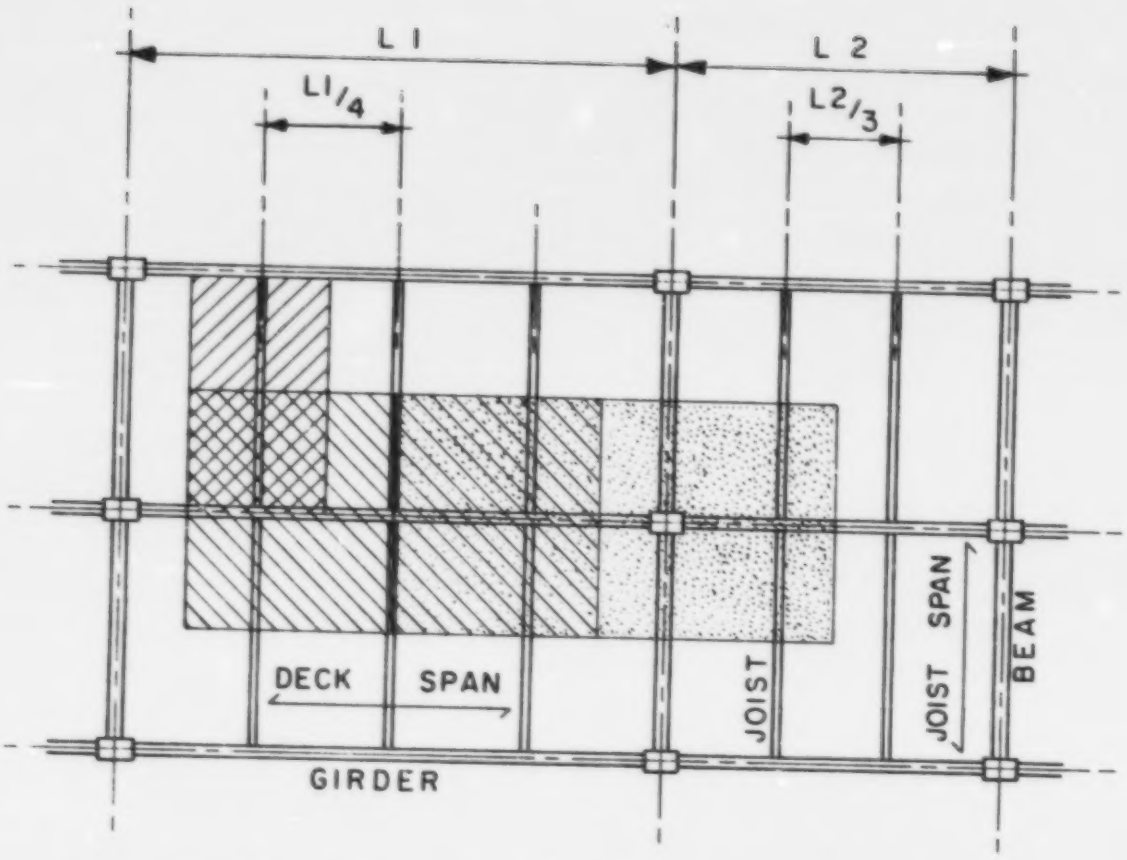


FIGURE 2 - Tributary Areas for One-Way Slab with Joists, Beams and Girders

Unbalanced Loading

Unbalanced loading cases should be considered in accordance with the National Building Code([2]) as well as the CSA concrete code([1]). The National Building Code 1980([2]) load considerations for snow load are as follows:

a) specified load distribution over the entire area;
b) specified load distribution over any one portion of the areas and half of this load on the remainder of the area, in such a way as to produce the greatest effects on the member concerned.

The CSA code CAN3 A23.3-M77([1]) requires load considerations for live loads on frames as follows:

a) full design load on two adjacent spans;
b) full design live loads on alternate spans.

It can be seen that the pattern loading requirements are not quite the same between each code. While it may be argued that snow load is not a live load under the terms of the building code, both sets of loading conditions should be examined.

Point Loads

Point loads applied to thin roof slabs may cause critical design stresses locally. Slabs should be checked for punching shear, beam shear and local flexure.

Punching Shear

Punching shear or two-way shear is characterized by diagonal cracking along the surface of a truncated cone or pyramid around the concentrated load. The critical section for punching shear is defined as one perpendicular to the plane of the slab, located so that its periphery is a minimum and at a distance of d/2 outside the periphery of the area of the concentrated load. This is shown diagrammatically in figure 3.

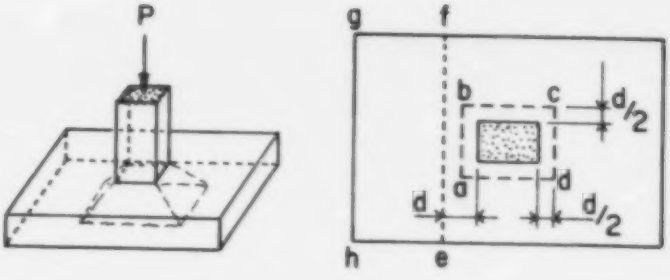


FIGURE 3 - Punching Shear

Beam Shear and Flexure

The analysis of the slab for beam shear and local flexure in an area affected by a point load is carried out by assigning an equivalent beam width, within the slab, which can be expected to distribute the point load to the supporting beams. The equivalent width 'bw' is shown diagrammatically in figure 4. The angle of radiation 'α' is dependent on several factors but the net variation in the value is

quite small. In general, a value of α = 15° is used for most analyses. The equivalent beam is analysed as a fixed-end beam with supports at the face of each supporting beam. The critical flexural sections are located at the face of the support for negative flexure and at the location of maximum positive for positive flexure.

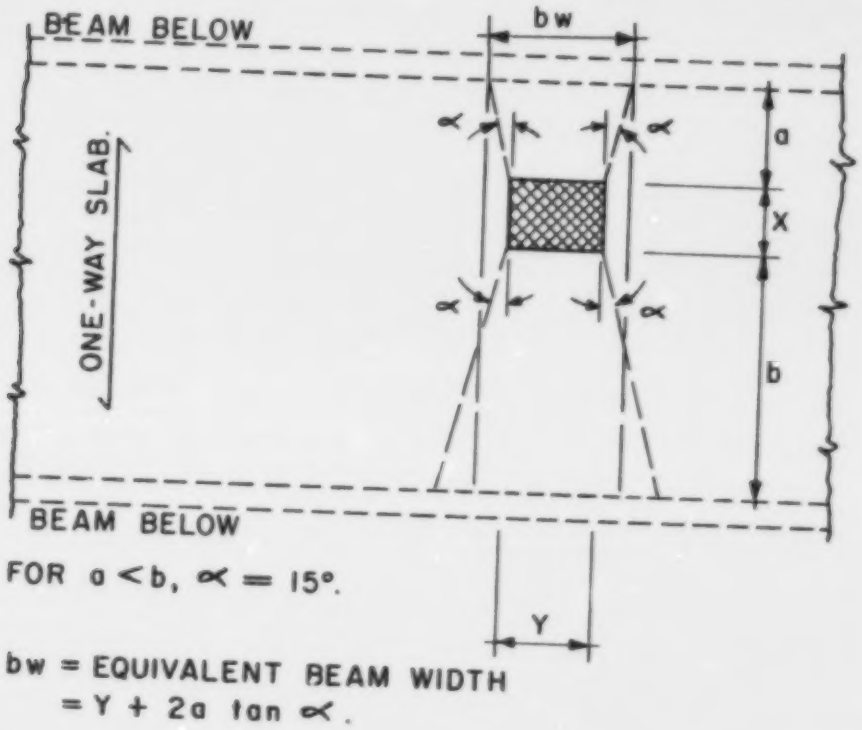


FIGURE 4 - Equivalent Beam Width for Point Load on One-Way Slab

Shear is calculated on the basis of diagonal tension, with the critical section for shear located a distance 'd' from the face of the beam.

For standard concrete floor joist construction the allowable shear stresses can be increased by 10%.

3.02 Spans

Simple Spans

Simple span members should be analysed using the lesser of the clear span + the depth of the member, or the distance between the centres of the supports.

Continuous Spans

Continuous spans should be analysed using the centre to centre distances between supports unless the spans are less than 3 metres. For spans less than 3 metres, the clear span should be used for analysis.

If centre to centre distances are used for the analysis, the critical section for negative design bending moment should be taken at the face of the support.

3.03 Analysis

The most advantageous analytical techniques to the structure, not to the engineer, should be used in the structural analysis. Continuous members should be considered so and either moment distribution techniques or computer analysis techniques should be employed to derive bending moments and shears. The approximate moment and shear formulae for frames given in the Concrete Code([1]) should not be used as a basis for the analysis of the structure. They are not sufficiently accurate.

Moment Redistribution

Moment redistribution should be used wherever possible to modify negative bending moments obtained by analysis. The negative moments may be increased or decreased by as much as $20\ (1 - \frac{p - p'}{p_b})$ percent. This formula is dependent on the ratio of the amount of negative steel to positive steel and is illustrated in figure 5. If negative moments are modified according to this formula, all other bending moments within the span must also be modified to maintain equilibrium. Thus, decreasing the negative bending moment at the support will result in increasing the positive bending moment in the span. The absolute amount of change in the negative moment will generally be the same as the amount of change in the positive moment, but the percentage change may be different depending on the base values of each bending moment.

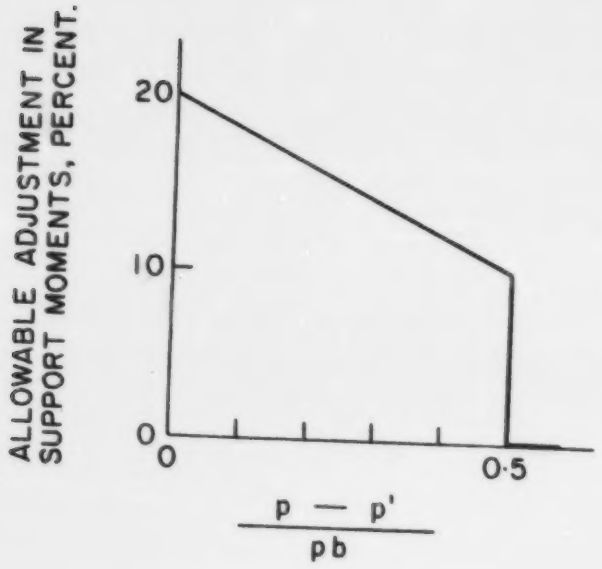


FIGURE 5 - Allowable Moment Redistribution

Critical Sections

Once bending moments and shears have been derived for the sturcture, it is then necessary to select the critical locations in the structure for design. This will usually result in both the bending moment and shear values which are less than the maximum values obtained in the analysis. The critical section for bending moment in beams and girders is located at the face of the support. The critical section for short slabs (less than 3.0 metres) is at the face of the support, but as noted in the span information, the design span is based on the clear span between supports. The critical section for shear is located a distance 'd' from the support.

T-beams

Girders and beams supporting one way slabs should be considered as T-beams for positive bending moment calculations. The wider compressive zone in the flange of the T-beam results in being able to consider a longer lever arm for the reinforced section, thus reducing the amount of positive reinforcement required. For negative moment calculations,

the longitudinal slab distribution steel may be considered as part of the negative flexural steel, provided such reinforcement is located within a distance 'b' each side of the beam web.

3.04 Code Changes

Minimum Reinforcement

In many older structures little consideration was given to using the minimum thickness of slab, beam or girder in the construction. This often resulted in the use of large sections which had small ratios of flexural reinforcement.

The Concrete Code([1]) sets limits on the minimum amount of flexural and shear reinforcement in sections to preclude abrupt failure. The limits are generally set such that the capacity of the cracked reinforced section with minimum reinforcement is stronger than the capacity of the unreinforced section at its cracking moment or shear limit.

Minimum Flexural Steel

Minimum reinforcement criteria for slabs have been included in the codes for many years but were apparently ignored by many designers of older buildings. For beams and girders minimum reinforcement criteria were introduced into the 1953 National Building Code([2]). The present code, CSA A23.3 M-77([1]) requires slabs to have a minimum reinforcing ratio of .0018 for 400 MPa rebar and .002 for rebar less than 400 MPa. Other flexural members such as beams and girders are required to have a minimum reinforcing ratio of $\frac{1.4}{fy}$ unless the area or reinforcement at every section is at least 1/3 greater than that required by analysis.

In many cases, slabs found in older buildings will be found to have less than the current specified minimum reinforcement. In such cases, the cracking moment of the slab should be calculated and compared with the reinforced cracked moment flexural capacity. If the section capacity is greater than the cracking moment, then the section may be

considered to be adequate. Otherwise, the section does not meet the safety intentions of the code.

Minimum Shear Reinforcement

The introduction of a minimum quantity of shear reinforcement for beams and girders was introduced in the 1960 National Building Code([2]). It basically requires steel to reinforce sections for shear stresses up to about .35 MPa.

Reinforcement Spacing

The maximum allowable spacing of reinforcing steel for flexure in slabs has been reduced over the years. Early codes permitted slab bar spacings up to five times the slab thickness or up to 600 mm. Current codes limit spacing to lesser of three times the slab thickness or 450 mm.

The maximum permissable spacing of stirrups for shear in beams and girders has also been reduced over the years. Early codes permitted stirrup spacings in beams up to 'jd' where 'j' could reach a value up to .9. Current codes limit stirrup spacing to .5d.

3.05 Composite Construction

A common method of building floors and roofs in the past was to use composite construction. A steel beam was cast partially into a concrete slab was considered to be a composite member. The analysis of the slab, perpendicular to the composite beam is not affected by recent code changes which require either the complete encasement of the beam or the use of shear connectors, but the beam itself is affected by these code changes.

Shear Connectors

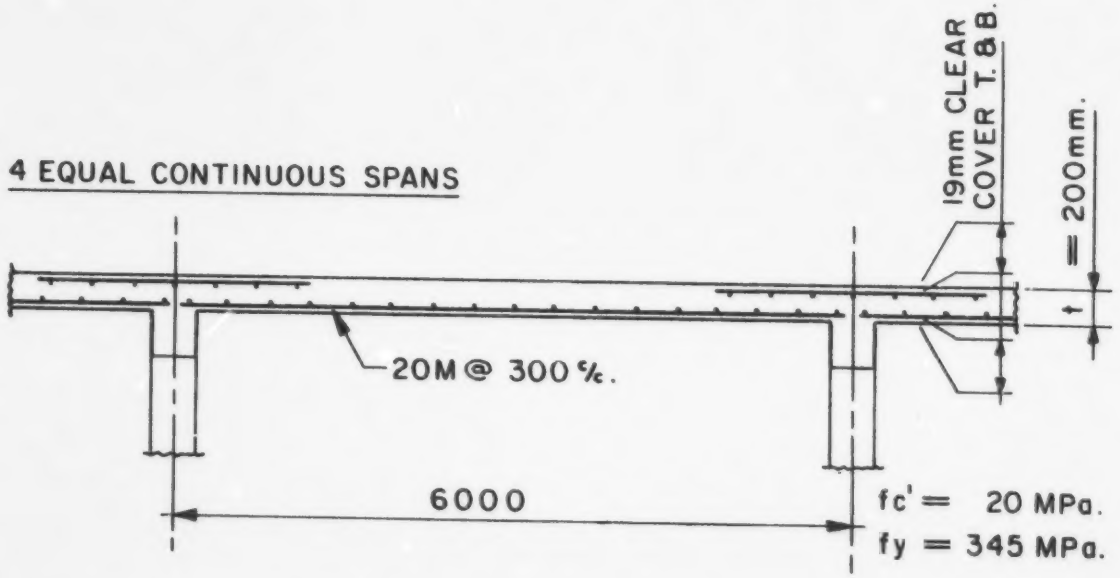
Before the 1965 National Building Code([2]) was issued it was permissable to consider that composite action occurred if a concrete slab was either encased or cast against the unpainted top flange of the beam. After 1965, positive methods were required to ensure composite action between beam and slab other than bond between the top

flange and the concrete. Accordingly, it is not uncommon to find steel beam and slab construction which does not appear meet current code requirements.

The introduction of more stringent requirements to composite action was considered necessary to avoid the possibility of an abrupt failure of the system. The capacity of the composite section, immediately prior to bond failure, is usually higher than the capacity of the steel beam alone. A slab with no positive shear connection could fail, without prior warning, as soon as bond failure occurred between the top flange of the steel beam and the conrete slab. Unfortunately, when considering such a system in which shear connectors have not been installed, the structure must be analysed as being non-composite.

3.06 Example

EXAMPLE #1 - Concrete One-Way Slab



BUILDING DESIGNED IN 1963 (TO 1960 NBC)
REROOFED IN 1978 WITH INVERTED ROOFING SYSTEM
PROPOSED INCREASING LIVE LOAD BY 10%

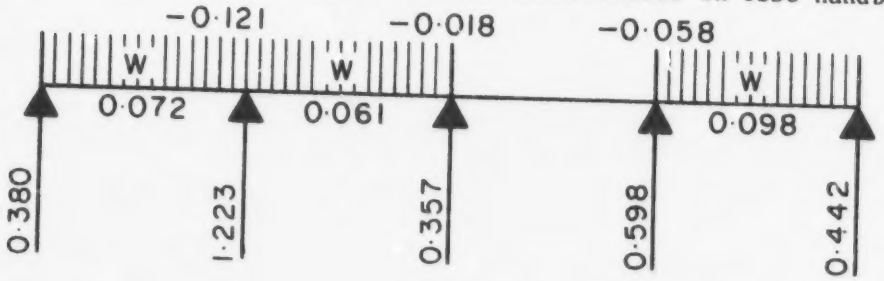
Loads

Dead - structure - 200 mm slab		4.79 kN/m^2/m
- roofing and insulation and ballast		
- existing membrane scraped clean		.24 kN/m^2/m
- new plies & insulation & ballast		.86 kN/m^2/m
- ceiling mechanical and electrical		.24 kN/m^2/m
	SUBTOTAL	6.13 kN/m^2/m
Live - snow based on (Cs = .8)		1.82 kN/m^2/m
	TOTAL	7.95 kN/m^2/m

STEP 1 - Calculate span length to be used for analysis

- Refer to CSA A23.3-M77 clause 6.6, 2.1
- use centre to centre of support unless actual span length less than 3000 mm
= 6000 mm - for design

STEP 2 - Analyse for maximum positive bending moment on interior span.
- refer to standard moment coefficients in CISC handbook ([7])



MOMENT = COEFFICIENT x W x L.
REACTION = COEFFICIENT x W.
WHERE: W = TOTAL UNIFORMLY DISTRIBUTED LOAD ON ONE SPAN.
L = LENGTH OF ONE SPAN.

M = .098 WL
= .098 x w.L^2

STEP 3 - Verify original design by WORKING STRESS DESIGN METHOD

W = 7.95 kN/m^2

M+ = .098 x 7.95 x 6.000^2 = 28.05 kN.M

$As = \frac{M}{fs\ jd}$ where $j = 1 - \frac{k}{3}$

$k = \sqrt{2np + (np)^2}$

STEP 3 (cont'd)

$d = 200\ mm - 20\ mm\ (cover) - \frac{1}{2} \times 20\ mm)(reba$

$= 170\ mm$

$n = 9.2$ for fc' = 20MPa

$p = \frac{As}{bd}$

$= \frac{1000}{1000 \times 170}$

$= .0059$

$k = \sqrt{2 \times 9.2 \times .0059 + (9.92 \times .0059)^2}$
$= .334$

$j = 1 - \frac{.334}{3} = .889$

$fs = .50fy$
$= .5 \times 345\ MPa$
$= 172.5\ MPa$

- must check crack widths otherwide use
fs = 24,000 psi

$MALL = As\ fs\ jd = 1000 \times 1725 \times .889 \times 170 \times \frac{1}{1000^2}$

$= 26.07\ kN.M$

MALL less than M+ and slab slightly inadequate

$WL = 7.95 \times \frac{26.07}{28.05} - 6.13$

$= 1.36\ kN.M/M$

STEP 4 - Check design capacity by Ultimate Strength Design

Load factors: $\alpha_D = 1.4$
$\alpha_L = 1.7$

Design loads: $Wud = 1.4 \times 6.13 = 8.58\ kN/m^2/m$
$Wul = 1.7 \times 1.82 = 3.10\ kN/m^2/m$

Slab capacity: $Mu = \phi\ (Asfy\ (d - a/2))$ where $a = \frac{As\ fy}{.85\ fc'b}$

$a = \frac{1000 \times 345}{.85 \times 20 \times 1000} \times \frac{1}{1000} = 20.3\ mm$

STEP 4 (cont'd)

$$Mu = .9\ (1000 \times 345)\ (170 - 10.15))\ \times \frac{1}{1000^2}$$

$$= 49.63\ \text{kN.M/M}$$

Ultimate uniform load: $Wul = \dfrac{Mu}{.098\ L^2}$

$$= \frac{49.63}{.098 \times 6.000^2} = 14.07\ \text{kN/m}^2$$

Ultimate live load: $Wul = Wu - Wud$
$= 14.07 - 8.58$
$= 5.49\ \text{kN/m}^2$

- Live load: $W_L = \dfrac{Wul}{1.7} = \dfrac{5.49}{1.7} = 3.23\ \text{kN/m}^2$

This is greater than 10% above the original design show load of 1.82 kN/m^2

Conclusion: The use of ultimate strength design methods allowed an existing slab which had previously been reroofed to accept and addtional 10% snow load.

4.00 TWO WAY SLABS AND BEAMS

A common concrete system used in past was on in which two way slabs were placed on beams or bearing walls. This system is not as common today due to the high cost of formwork for the beams and the low cost of concrete and reinforcing steel for the slabs.

4.01 Code Changes

Before the publication of the 1975 National Building Code([2]) (CSA Standard A23.3-1973([1])) two way slabs with beams were afforded a separate section in the code and were considered to be different than both one way slabs and flat slabs. Since then two-way slabs have been included in the section on the design of flat slabs, reflecting the similar continuity and compatability aspects in both the systems.

The analysis of two-way flat slabs and beams is therefore dealt with in the section on flat slabs in the manual.

5.00 FLAT SLABS

Flat slabs include all slab systems reinforced for flexure in more than one direction, with or without beams between supports. They also includes waffle slab construction and slabs with panelled ceilings.

Analysis

There are two empirical analytical techniques in common use for the analysis and design of flat slab systems. These are outlined in CSA Standard A23.3-M77([1]). They are:

a) The Direct Design Method
b) The Equivalent Frame Method

Both these techniques are considered empirical because they model three dimensional action using a two dimensional frame. More sophisticated techniques such as three dimensional finite element analysis or yield line theory may be used, but such use is not common in the analysis of ordinary buildings. For the purposes of this manual, the two empirical methods only are considered.

Direct Design

A form of the Direct Design Methods has been included in the National Building Code since its inception in 1942. The Direct Design Method has been modified through the years to its present form today, but has always maintained a simplicity which allowed the work to be undertaken by hand, without a continuity analysis of the structure.

Equivalent Frame

An elastic analysis type of solution for slab systems was first introduced in Canada in 1953 National Building Code. This elastic solution has been modified considerably since that time to its present form, the Equivalent Frame Method. The elastic analysis has always

been a more time consuming technique to undertake by hand. However, with the adaption of the Equivalent Frame Method to the high speed digital computer in the early 1970's, the equivalent Frame Analysis has become pre-eminent.

5.01 Loads

Both uniformly distributed loads and concentrated point loads should be considered when analysing flat slab structures.

Uniform Loads

Live Load Reduction

No live load reduction factors should be applied to uniformly distributed snow loads for flat slabs being considered for solar collector installations. Reduction factors are applicable only to "use and occupancy" live loads and not to snow loads.

Tributary Area

The tributary areas for uniformly distributed loads, applicable to slab systems, are: "areas bounded by the lesser of column lines or a combination of column lines and lines of supporting members such as beams and girders".([3]) Such areas are shown in figures 6, 7 and 8.

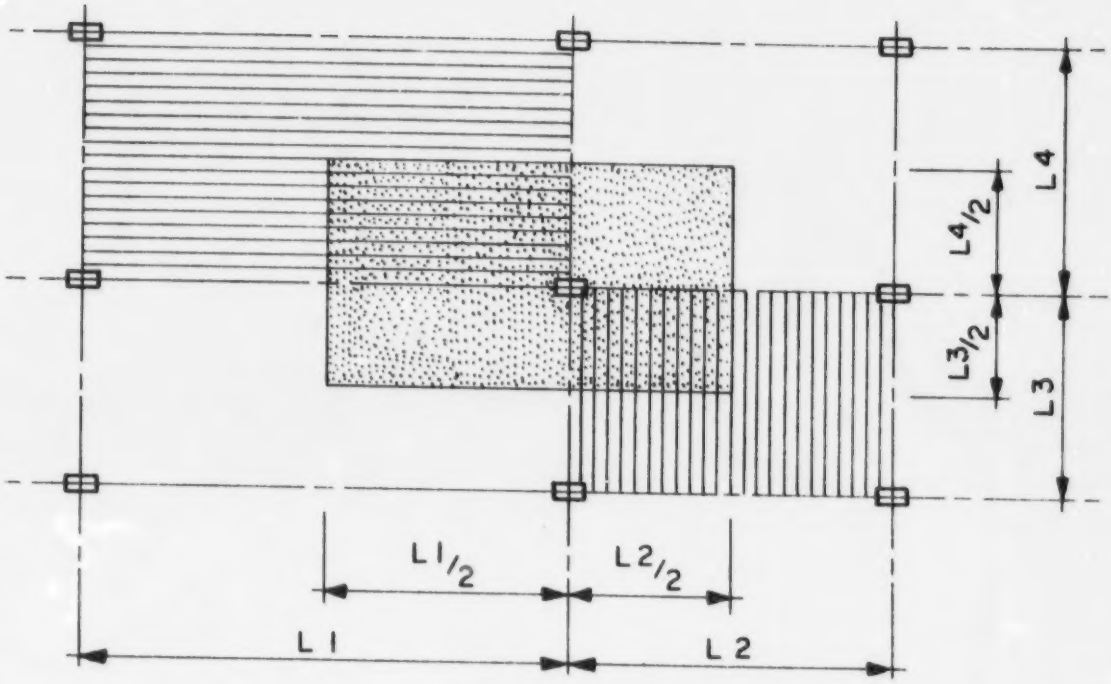


FIGURE 6 - Tributary Areas for Flat Slabs without Beams and Girders

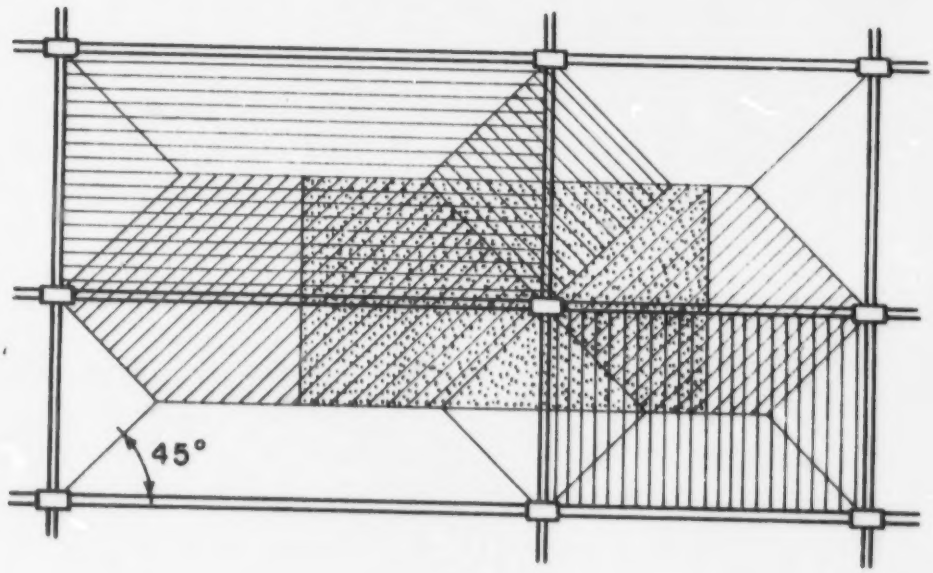


FIGURE 7 - Tributary Areas for Two-Way Slabs with Beams

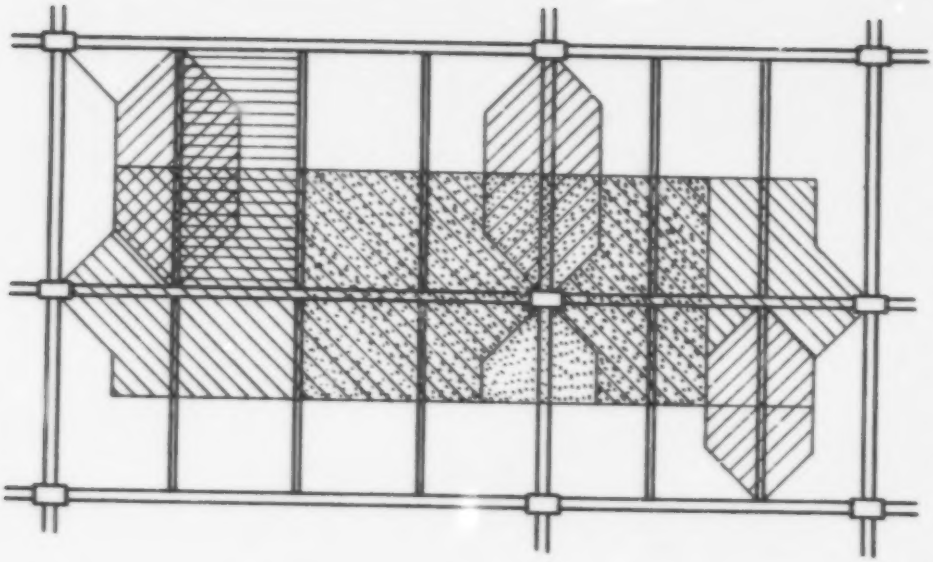


FIGURE 8 - Tributary Areas for Two-Way Slabs
Joists, Beams and Girders

Unbalanced Loading

Unbalanced loading conditions for uniformly distributed loads should be considered in accordance with the 1980 National Building Code ([2]) and CSA Standard A23.3-M77 ([1]). The combination for each code is slightly different, when considering snow load conditions on a roof. The National Building Code 1980 ([2]) requires the following be considered.

a) specified load distribution over the entire area;
b) specified load distribution over any one portion of the area and half of this load on the remainder of the area, in such a way as to produce the greatest effects on the member concerned.

The CSA Concrete Code ([1]) specifies the following loadings be considered when the actual conditions are unknown:

a) provided the live (snow) load is less than ¼ of the dead load, full design live load on all panels;
b) with live load greater than ¼ of the dead load;
   i) full design live load on all panels;
   ii) ¾ design live load on alternate panels;
   iii) ¾ design live load on two adjacent panels.

It can be seen that the pattern loading conditions for each code are somewhat different and hence all loading conditions should be considered.

Point Loads

Point loads are generally not as critical to flat slab systems as to one-way slab systems, because flat slabs tend to be much thicker.

For flexure, point loads are normally considered to be spread out into a uniformly distributed load of limited area.

Punching shear is checked based on the characteristic diagonal cracking along the surface of a truncated cone or pyramid around the concentrated load. The critical section is the same as that described in section 3.01 for one-way slabs and beams.

5.02 Spans

For analysis of a slab system by the "Direct Design Method", bending moments and shears are calculated based on the clear span of the slab, face to face of supports. For analysis by the "Equivalent Frame Method", spans are defined as the centre to centre distance between supports.

5.03 Analysis

A choice of analytical methods, the Direct Design Method or the Equivalent Methods, must be made. Where geometry is irregular, however, only the Equivalent Frame Method may be used. If either method can be used it is important to realize that the analytical results produced by each method, while similar, are not identical.

Direct Design vs. Equivalent Frame

Slab systems analysed by the Direct Design method generally produce results with lower negative bending moments and higher positive bending moments. Slab systems analysed by the Equivalent Frame Method yield results having higher negative bending moments and lower positive moments. These results seldom differ by more than 10% but they may be significant in re-analysising an existing structure for some slight increase in loading. This is especially so if the original design were carried out using one method and the reanalysis carried out using a different method.

Equivalent Frame Method

The Portland Cement Association developed a computer program based on the equivalent frame method in the early 1970's, which efficiently analyses and designs flat slab systems, with or without built-in beams. The program has been updated several times. This program has been updated several times. The current program is called ADOSS ([8]) and it is available from most time-sharing computer system houses at a very modest cost to run. This latest version of the program incorporates all the provisions of the ACI Concrete design code ([9]) to optimize the slab design. The ACI requirements are very similar to those of CSA A23.3-M77 ([1]).

Many consulting firms have developed their own programs for analysing slabs based on the Equivalent Frame Method. Should analysis by the Equivalent Frame Method be considered appropriate, the best way to approach the problem would be to use one of these sophisticated programs to do the work. If this is not possible, the Portland Cement Association ([4])([5]) have also published a book of notes with design applications which carries through a step-by-step solution of a sample Equivalent Frame Method solution of a flat slab structure.

Direct Design Method

The Direct Design Method is more suitable to a hand of analysis although the work involved in solving a single bent is considerable. The Portland Cement Association book of "Notes on ACI 318-71 Building Code Requirements with Design Applications" ([4])([5]) also carries

through a worked example, step-by-step, of a sample Direct Design Method solution of a flat slab. The format and solution can be used as a model for other similar hand calculations.

In carrying out a hand solution using either Direct Design or Equivalent Frame methods, certain key Code items should be noted which can afford some relief to a structure loaded close to its limits:

a) Bending moments for negative flexure should be calculated at the critical section, rather than at the centre-line of the support. This results in a considerable reduction in the design bending moment. In general, the critical section is located at the face of the rectilinear support, but because there are differences between the Direct Design and Equivalent Frame Methods, the specifics in the Code should be noted.

b) The total shear contributing to punching failure at the column may be reduced from the total calculated shear on the panel by subtracting the load associated with the area of slab supported directly over the column within the perimeter of the area defined as the critical section for punching shear.

Moment Redistribution

Both analytical methods for flat slab systems yield design shears and design bending moments for critical sections. Under the provisions of the Code ([1]), these design bending moments may be modified by up to 10% provided modifications are consistent. This permissable modification is to account for possible moment redistribution. It should be noted that, in general, computer solutions do not incorporate any moment redistribution. Redistribution must be done by hand after the computer results are obtained.

By selecting the appropriate analytical technique (direct design vs. equivalent frame) and applying moment redistribution, bending moments differing by up to 25% may be obtained at any one section between the two methods.

5.04 Code Changes

Code changes mentioned previously in section 5.03 Analysis which concern the evolution of the analytical methods for flat slabs will not be repeated here. Certain other code changes however, also affect reanalysis of an existing slab.

Minimum Flexural Reinforcement

In the past many designers placed steel in the middle strips of flat slabs without regard to minimum steel requirements. Instead, bending moment was assigned to the middle strips and steel was designed based on these moments. Unfortunately, all to often, the bending moments in the middle strips were calculated to be very low and inadequate steel was placed in the strips. The present minimum steel requirement is .18% to .20% of the gross section depending on the grade of steel used. Where deficiencies in minimum steel re uirements occur, the cracking moment of the slab should be calculated to ensure that it is lower than uncracked moment capacity of the reinforced section.

Unbalanced Moment Transfer

In 1965 the concept of unbalanced moment transfer from slab to column was introduced into the Code. Structures designed prior to that time may exhibit weaknesses at the column/beam junction as far as punching shear and moment transfer are concerned, especially at the end columns and first interior columns where unbalanced moments are highest.

6.00 ADVANCED ANALYSIS TECHNIQUES

A discussion on the detailed analysis of concrete structures would not be complete without mentioning some of the most advanced techniques which could be used. The advanced analysis methods which may be applicable to reinforced concrete design include:

a) Yield-line analysis and Limit analysis;
b) Finite Element analysis.
c) Load testing

The first two methods yield upper bound solutions which are non-conservative so care should be excercised in their application as related to the safety of the structure. The load test is the most accurate method of structural analysis there is as the structure itself performs the complicated distribution of loads to the various parts of the structure.

Yield Line and Limit Analysis

Yield-line theory and Limit Analysis are based on an ultimate theory of failure rather than the traditional elastic behaviour of the structure. Both are methods of moment analysis, based on realistic considerations, which direct attention to conditions that occur in the the structure just prior to failure. These methods permit the determination of failue moments in structures of irregular shape for a variety of loadings. Yield line analysis is particularly relevant to slabs whereas limit analysis is usually applied to frame-type structures.

Both methods involve the concept of the formation of "plastic hinges" in the more highly stressed regions. These "plastic hinges" allow rotation of the structure about the hinge at a constant moment. As the hinge rotates, providing a constant restraining moment to the structure, other locations become more highly stressed to the point where other hinges occur. When sufficient hinges have formed, a mechanism for failure occurs and the structure is deemed to have failed. While the theory sounds simple, the analysis of a structure for a single failure mechanism is quite difficult. Then a multitude of failure mechanisms must be examined to determine which will cause failure at the lowest possible load.

Finite Element Analysis

Finite element analysis of a structure involves the modeling of the various components of the structure, each by small elements, then combining all elements in a computer and carrying out a stiffness or flexibility analysis of the model. The logistics of breaking down a concrete structure to model it as finite elements are considerable. Certain large scale finite element computer programs possess grid and element generators to aid in part of the solution.

The choices of element type and element size play large roles in the accuracy of the results. The correct type of element to model a certain part of the structure should be available and to achieve any degree of accuracy, it should be as small as possible. There is, however, a practical limit to the number of elements the computer program can handle before arithmetic error inherent in the solution method exceeds the accuracy optainable by reducing the element size and thus increasing the number of elements in the model.

While limit type analysis (yield line) and finite element analysis are probably more accurate than techniques assessed in the first five sections of this chapter, generally their cost, both in engineering time and computer time, render then too expensive to be practical for conventional building analysis for solar collector installations.

Load Testing

Load testing a structure which appears to be under capacity analytically or one for which there is not adequate information or budget to obtain information for an analysis is often a means which will permit additional loads to be approved for the structure.

Heavy structures, such as reinforced concrete and steel beam and slab often have considerable reserve capacity which cannot be calculated analytically. Light structures generally have little extra capacity becuase of the flexibility and of redundency.

The cost of a load test should be compared to the cost of a detailed engineering analysis or the cost of reinforcement to upgrade the structure before proceeding with any of the options, especially if there is a general feeling of confidence in the structure.

Load testing of concrete structures should be carried our in accordance with the terms of chapter 18 - Strength Evaluation Procedures of CSA Standard CAN3 A23.3-M77 ([1]).

Common materials used to impose the test loads on the structure include pallets of concrete blocks or vinyl lined swimming pools. Shoring beneath a load test is a necessity.

7.00 REFERENCES

1. CSA Standard CAN3-A23.3-M77 "Code for Design of Concrete Structures for Buildings".

2. The National Building Code 1942, 1948, 1953, 1960, 1965, 1970, 1975, 1980.

3. "Commentary G", The Supplement to the National Building Code of Canada 1980.

4. "Notes on ACI 318-71 Building Code Requirements with Design Applications" Portland Cement Association, 1972.

5. "Notes on ACI 318-77", Portland Cement Association, 1978.

6. "Design Handbook in Accordance with Strength Design Method of ACI 318-77 Vol. 1 - Beams, Slabs, Brackets, Footings and Pile Caps", ACI Committee 340.

7. "Handbook of Steel Construction" Third Edition, Canadian Institute of Steel Construction, 1980.

8. "Analysis and Design of Slab Systems" Program Users Manual, Portland Cements Association, 1976.

9. "Building Code Requirements for Reinforced Concrete - ACI 318-77" American Concrete Institute, 1977.

CHAPTER 6

DETAILED ANALYSIS OF STEEL STRUCTURES

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DETAILED ANALYSIS OF STEEL STRUCTURES

1.00 INTRODUCTION

Structural steel design has grown out of the design of iron work in the last century. The materials used in iron/steel work have changed from cast iron to wrought iron to high carbon steel to low-alloy steel. The strengths of steel employed in the building construction industry have increased substantially since the turn of the century. But many of the buildings constructed early in the century are with us today, in need of upgrading or renovating, or to have new solar installations added.

The increase in the steel strengths used in construction has been matched by a growing sophistication in both the design and the construction fields, thus resulting in an improved efficiency in the overall construction process. The Codes governing the design of steel structures have become larger and more complex as the body of knowledge concerning the behaviour of structures has grown. A major change in the design philosophy for steel structures was introduced in Canada in 1974 - the inception of a "Limit States Design" approach.

This method is expected to gradually replace the traditional working strength or allowable stress design approach. Plastic design methods, in lieu of elastic design methods have also been recently introduced. However, there has been less of a tendency to move from elastic to a plastic analysis in steel. Steel tends to remain in an elastic state for most of its servicable-load life and the cost of ensuring that plastic hinges can occur without premature buckling failure generally raise the cost of a plastically designed structure above that of an elastically designed one.

Inevitably, with greater material strengths, more sophisticated analytical techniques and removal of redundancies in construction by the design and construction processes, safety factors in certain areas

of building decreased to a point where additional clauses have had to be inserted into the Code to prevent premature failure of the structure. Bracing requirements have become more prominent in the Codes. Clauses to prevent premature failure of structural elements have also had to be introduced. And, servicability (deflection and vibration) criteria have been revised considerably to reflect loss of load sharing and significant increases in the span/depth ratios used in construction.

This chapter deals with only the horizontal load carry elements of a structural steel frame, such as metal decking, joists, beams and trusses. Axial load carrying members, such as columns, are not treated herein because they are seldom found to be critical elements in the analysis of structures for the installation of solar collectors. In general, the flexural strength of the horizontal load carrying elements in steel structurals is the critical design factor. Shear, except in joists, does not usually control.

2.00 LIMIT STATES DESIGN

The "Limit States Design" method was introduced in 1974. It is similar to the "Ultimate Strength Design" method for reinforced concrete in format, but the load factors and performance factors are quite different in magnitude.

Depending on the nature of the structure, analysis using the "Limit States Design" method alone may be all that is necessary to permit increasing the safe design loads on the structure.

Use

Unlike the concrete code which no longer recognizes allowable stress design, the current CSA Standard S16.1-M77([4]) recognize both Allowable Stress and "Limit States Design" methods. While both techniques are recognized, the two methods should never be used together for analysis. For analysing an existing structure, for the installation of solar collectors, the "Limit States Design" method should always be used.

Current Load Factors

The current load factors specified in "Limit States Design" are:

1.25 for dead loads

1.50 for live loads

In combinations where two or more types of live loads act simultaneously, provisions have also been made for reducing load effect, because of a reduced probability of simultaneous occurrence of more than two live loads.

Future Load Factors

Unlike the concrete load factors, the load factors for steel design are not expected to change significantly in the near future.

3.00 STEEL JOIST CONSTRUCTION

This section includes open web steel joist with V-pan forming and concrete deck as well as steel joist construction with non-cellular metal roof deck. Light structural systems, such as those incorporating steel joists with either metal deck or thin concrete slabs, seldom have strength reserves which can be realized by carrying out a detailed analysis.

3.01 Loads

At the detailed analysis stage all dead and live loads should have been reduced to the minimum acceptable level, considering both the actual conditions and the minimums specified in the Codes.

Uniform Loads

Live Load Reduction

Commentary G of the Appendix to the 1980 National Building Code(6) states "No live load reduction factors should be applied to wooden or sheet metal decks ..... because of the uncertainty of the degree of lateral distribution of loads". In general, the tribuary area

associated with a piece of deck or a single joist is too small to warrant live load reduction. In any case, live load reductions are not applicable to snow loads.

Unbalanced Loadings

Unbalanced loading conditions should be applied to joist systems. It is a requirement of the Building Codes([5]). Unbalanced loadings can cause stress reversal in some web members of the joists. The National Building Code 1980([5]) states that the following unbalanced loadings for snow should be considered:

1) specified load distribution over the entire area;

2) specified load distribution over any one portion of the area and half the load on the remainder of the area in such a way as to provide the greatest effects on the member concerned.

CSA S16.1-M77([4]) requires the following uniform loadings for joists be considered:

1) A uniformly distributed load equal to the total dead and live load.

2) An unbalanced load with 100 percent of the total dead and live load on any continuous portion of the joist and 25 percent of the total dead and live loads on the remainder to produce the most critical effect on any component.

There are clearly differences between the two codes for unbalanced loading effects. All conditions should be examined.

Distribution of Uniform Loads

When analysing joists, uniform loads should be converted to concentrated loads, applied at the panel points, for the determination of axial loads in members.

Flexural effects in the top chord due to uniform loading may be ignored provided the panel spacing is less than 610 mm. Otherwise, the effect of uniform loads on the top chord must be considered to determine the bending moment in the chord. Flexural effects must then be considered to occur simultaneously with the axial forces.

Concentrated Loads

As noted previously, uniform loads should converted into concentrated loads, applied to the panel points whenever possible. Concentrated loads, which occur naturally, should be applied to the joists where they occur. The flexural effect, caused by a concentrated load applied other than at a panel point must be assumed to occur simultaneously with the axial force in the member regardless of panel point spacing.

3.02 Spans

This section deals with both the overall length of the joist as well as the effective length of the individual members comprising the joist.

Main Joist Span

Either of the following spans should be used in the analysis of a joist:

1) The clear span, face to face of support +65 mm.
2) The end panel point to end panel point distance. The end panel point is defined as the intersection of the neutral axes of the top chord and the end web diagonal. The end connection or joist shoe must be checked for eccentricity. The joist span, calculated in this manner will often tend to be shorter than the clear span +65mm. It should therefore be used by preference.

Effective Length

Knowledge of the effective length of the compression members of a joist is necessary for proper analysis. Compression members include

the top chord and some of the web members. The bottom chord may occasionally also be a compression member if the joist is cantilevered or if its bottom chord is continuous into the support.

The effective length (KL) of the compression chord depends on the axis of buckling:

1) For X-X (horizontal) axis bending, Lx is the distance centre to centre of the panel points.
   K = .9 The panel points are defined by the intersection of the neutral axes of the web members with the chord members.

2) For Y-Y (vertical) axis bending, Ly is the distance centre to centre of the attachments of the deck.
   K = 1.0.
3) For Z-Z (skew) axis, Lz is the distance centre to centre of the panel points or spacers or both.
   K = .9. Decking should not be considered to fulfill the function of batten plates or spacers for top chords.

The effective length of web members is equal to the member length, centre to centre, between the intersection of the axes of the web and the chord members.
K = .9 for buckling in the plane of the web.
K = 1.0 for other cases.

3.03 Analysis

Model

Normally, open web steel joists are analysed as fully laterally supported pin-jointed trusses with loads applied at the panel points. Only axial forces are considered in the component members unless:

1) the top chord panel point spacing exceeds 610 mm;
2) concentrated loads are applied at locations other than panel points.

Top Chord Bracing

To determine the effective length of the top chord about the Y-Y axis for analysis, proper evaulation of the top chord bracing is necessary. To provide adequate top chord lateral support, individual bracing members must be capable of resisting 1% of the maximum axial force in the chord. Continuous bracing, such as provided by concrete or steel deck must resist 5% of the maximum axial force in the chord distributed uniformly over the length of the member.

Many older structures using thin concrete deck slab poured on metal V-ribbed forming utilize a lateral support system for the joists compresing wooden wedges driven through the metal pan and between the two members of the joist's top chord. Spacing of the wedges ranges from about 600 mm to 1000 mm. Careful evaluation of such a system is required to ensure that these connections, in total, can resist 5% of the maximum joist chord force.

Metal decks which are welded to the top chord of the joist or concrete slabs cast directly onto the top chord generally provide adequate top chord bracing.

Bridging

There are two kinds of bridging, utilized in open web steel joist construction; horizontal bridging and diagonal bridging. The purpose of bridging is to:

1) support the joists against lateral moment during construction;
2) hold the joists vertical or in a specified plane during construction;
3) limit slenderness ratios for chord members.

Once the structure is erected, the joists have been supported against lateral moment and have been held in a specified plane during construction. Therefore, when evaluating an existing structure, the first two requirements have already been satisfied. The third

requirement, to limit chord slenderness ratios, applies both during construction and after the building is completed. Top chord slenderness ratios are governed by the need to achieve a reasonably sized design section. This is achieved by fastening the chord to the metal deck or slab rather than by installing bridging. Therefore, in an existing structure with good connections between the top chord and the slab or deck, top chord bridging is no longer necessary. Bottom chord bridging, however, is still required to limit bottom chord slenderness ratios in an already constructed building.

Bridging in older structures, especially those utilizing closely spaced joists (about 600 mm centres), is often inadequate with respect to bottom chord bridging. The bridging may not exist. It may be too widely spaced. Or it may only be wired in place. In these cases, consideration should be given either to welding the existing bridging in place or to using the existing ceiling furring (metal channels) as a substitute for bottom chord bridging.

3.04 Code Changes

Spacing

Before the 1948 National Building Code([5]) came into effect, the spacing of open web steel joists was limited to a maximum of 750 mm. In the 1953 Code([5]), greater spacings were permitted provided the joists were designed as trusses. In the 1960 National Building Code ([5]), all restrictions were removed on the spacing of joists.

Lateral Support of Compression Chords

In the 1960 National Building Code([5]), the first specific requirement to provide lateral support to the top chord was introduced. It required bolts, welds or wedges to resist a total lateral force of at least 20% of the vertical load on the joist.

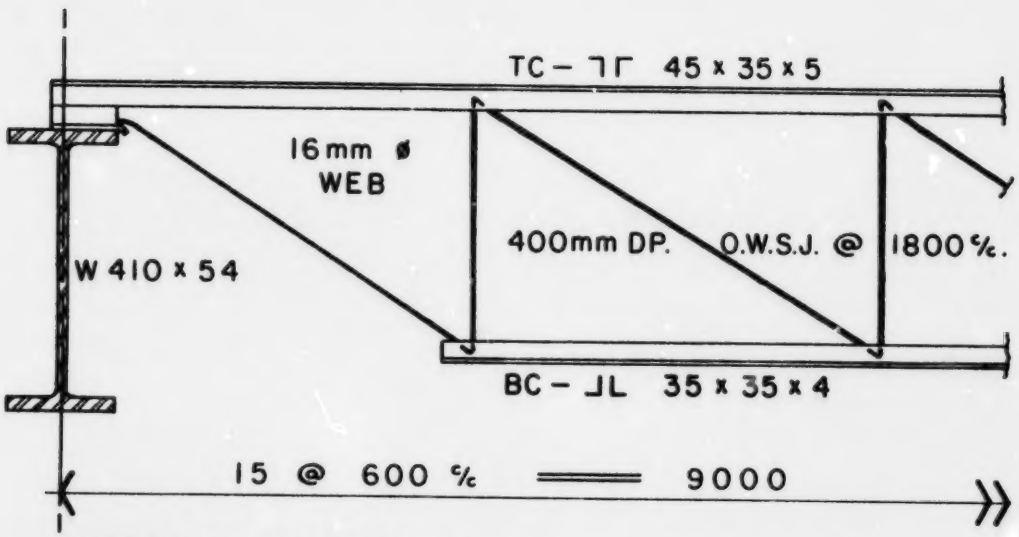
In the 1965 National Building Code([5]), top chord lateral support by metal decking was required to resist 5% of the maximum force in the top chord. This is similar to current Code requirements.

Compression Chord Bending

Bending in the top chord of an open-web steel joist has been recognized in a general way as a significant design requirement in open web steel joists since the 1948 National Building Code. Before 1960 only concrete slabs with a thickness greater than 1/12 of the panel spacing were assumed to be rigid enough to permit uniform loads to be redistributed as concentrated distributed to the panel points. The 1960 National Building Code([5]) introduced the specific recommendation that the flexural effect in top chords could be neglected, provided panel point spacing was less than 610 mm.

3.05 Example

EXAMPLE 1



CHORDS..... fy = 379 MPa.
WEB..... fy = 303 MPa.

DESIGN LOADS

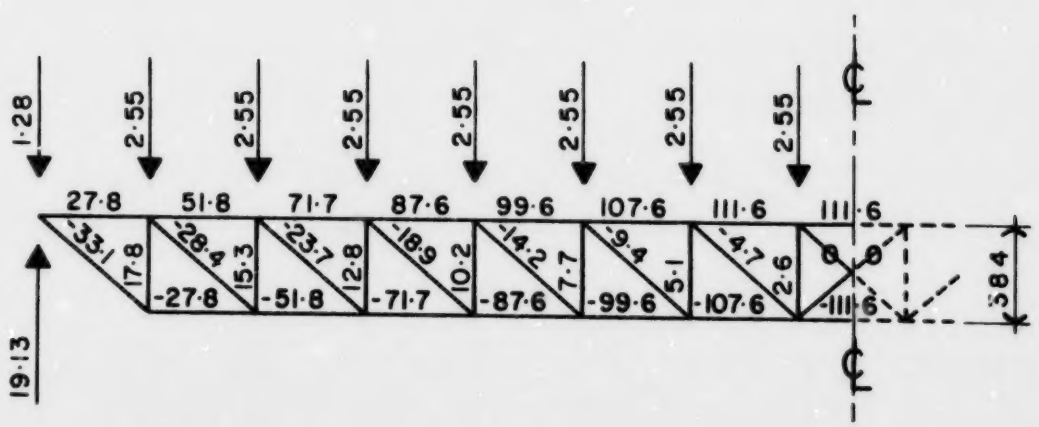
		Allowable		Factored
DEAD LOAD:				
ROOFING		.34	kN/m^2	
METAL DECK		.10	"	
MECH & ELEC		.10	"	
JOISTS		.10	"	
	TOTAL	.64 kN/m^2		.80
LIVE LOAD:				
SNOW (C_s = .8		1.72 kN/m^2		258
	TOTAL	2.36 kN/m^2		338

STEP 1 - Analyse by TOTAL stress design

2 - 45 x 30 x 5L A = 700 mm^2
Rz = 6.5 mm
Y = 7.9 mm

2 - 35 x 35 x 4L A = 528 mm^2
Rz = 10.2 mm
Y = 7.4 mm

Depth of Section
400 mm - 8.3 - 7.4 = 384.3 mm



TOP CHORD

$$\frac{KL}{Rz} = \frac{.9 \times 600}{6.5} = 83.0$$

Fa = 149 MPa

Pa = 149 x 700 x 1/1000 = 104.3 kN

PT/C = 111.6 kN

Therefore top chord is just under capacity

BOTTOM CHORD

Pa = .6 x Fy x As

= .6 x 379 x 528 x 1/1000 = 120.1 kN

PB/C = 116.3 kN

Therefore chord is almost exactly "on capacity"

END DIAGONAL

$$Pa = .6 \times Fy \times As = .6 \times 303 \times 20^2 \times \frac{\pi}{4} \times \frac{1\ kN}{1000} = 57.1\ kN$$

Pd = 33.1 kN

Therefore end diagonal is adequate

GOVERNING CONDITION

The top chord governs the design.
The equivalent uniformly distributed load is:

Weq = 114.3/111.6 x 2.36 kN/m^2

= 2.42 kN/m^2

Wlive = 2.45 - .64 kN/m^2

= 1.78 kN/m^2

STEP 2 - Analyse by limit states design.

BOTTOM CHORD

Pu = Ø x As x fy

= .9 x 528 x 379 1/1000 kN

= 180.1 kN

$$Pf = \frac{1.25 \times .64 + 1.50 \times 1.72}{2.36} \times 111.6$$

= 159.8 kN

STEP 2 (cont'd)

TOP CHORD

This is class 3 section

$$\lambda = \frac{KL}{r}\sqrt{\frac{Fy}{\pi^2 \times E}} = \frac{1.00 \times 600}{8.5} \times \sqrt{\frac{379}{\pi^2 \times 207 \times 10^3}}$$

$$= .986$$

$$CR = \phi A \times Fy \times (1.035 - .202\lambda - .222\lambda^2)$$

$$= .9 \times 700 \times 379 \times (1.035 - .202 \times .96 - .222 \times .96^2) \times 1/1000)$$

$$= 151.9 \text{ kN}$$

Pc = 159.8 kN (previous calculations)

END DIAGONAL

$$Pu = \phi \times As \times Fy$$
$$= .9 \times \frac{\pi^2}{4} \times 20^2 \times 379 \times 1/1000 \text{ kN}$$

$$= 107 \text{ kN}$$

$$Pl = \frac{1.25 \times .64 + 1.5 \times 1.72}{2.36} \times 33.1$$

$$= 47.4 \text{ kN}$$

GOVERNING CONDITION

The top chord governs the design.
The equivalent uniformly distributed load is as follows:

Wueq = 151.9/159.8 x 3.38 = 3.21 kN/m^2
Wulive = 3.21 - .80 kN/m^2 = 2.41 kN/m^2
Wlive = 2.41/1.5 = 1.61 kN/m^2

CONCLUSION

The joist has somewhat less capacity when analyzed by limit state design rather than working stress design.

4.00 STEEL BEAM CONSTRUCTION

This section covers all structural systems utilizing steel beams as primary framing members. It also includes those structures where steel joists or light steel beam purlins are used as secondary members in conjunction with metal roof deck or concrete one-way slabs.

In many cases, because of the discrete sizing of steel beams, there has been some reserve capacity already designed into the structure. "Limit States Design" methods of analysis favour the analysis of existing structures which have a high dead load to live load ratio.

4.01 Loads

Concentrated

Roof snow loads are normally specified as uniformly distributed loads. These uniform loads should be applied, as is, to the secondary framing. The reaction from the secondary framing should be applied to the primary framing members as concentrated reaction rather than redistributing the reaction back into uniform loads.

Consider a standard roof with 38 mm deck spanning to 400 mm deep steel joists spaced at 1750 mm centres. The joists span 7600 mm to simply supported steel beams. The uniform snow loading on the deck is applied to the joists, by assuming the uniform load acts as a series of point loads applied to the joist panel points. The load is transferred from the secondary joists to the primary beams at the joist seats. The joist reactions should be considered as point loads acting at each joist seat rather than as a uniformly distributed load on the beam. The bending moment in the beam is 5% higher if the joist reactions are considered to be a uniform load distribution rather than as concentrated loads applied at joist shoe locations.

Tributary Areas

If steel beams are used as secondary framing members (purlins), uniform loads should be applied in accordance with the requirements of Appendix B of the 1980 National Building Code(6). Generally the

tributary areas to purlins are not large enough that live load reduction factors may be applied and in any case, reduction factors are not applicable to snow loads.

Where steel beams are primary framing members, the tributary areas may be large enough (greater than 20 m^2) to qualify for general live load reduction unless the occupancy is storage, manufacturing, retail, garage or assembly. Snow loads are not, however, considered as live loads and cannot be considered for reduction.

Unbalanced Loads

The requirements for unbalanced loading conditions as specified in both the National Building Code([5]) and CSA Standard S16.1-M77([4]) should be incorporated into the analysis of steel beam systems. If the beam are constructed as simple spans rather than continuous ones, unbalanced loading effects from adjacent frames do not exist. If spans are continuous, unbalanced effects be considered.

Unbalanced loading conditions must be considered when analysing semi-continuous, or gerber-type construction. The application of full live load on all spans governs the positive moment design in the drop-in section as well as in the negative moment design of the cantilever section. The pattern loading condition, with full live load on the cantilever beam main span and reduced live load on the adjacent spans governs the positive moment of the cantilever span. This is illustrated in figure 1.

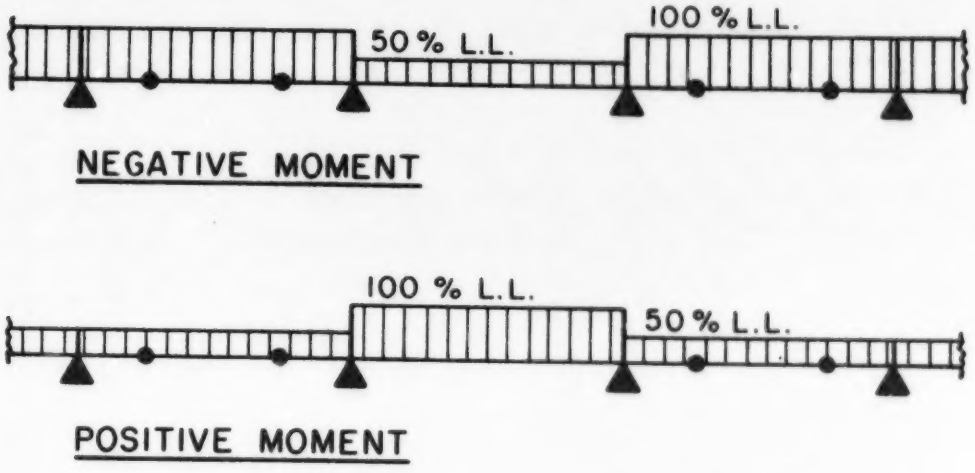


FIGURE 1 - Unbalanced loading conditions for Semi-Continuous Construction

4.02 Spans

Continuous Construction

When analysing continuous structures having steel beams as primary framing members, the span length is the centre to centre distance between the centres of gravity of the supporting members. Span lengths for Semi-continuous construction (gerber-type) should also be calculated in a similar manner.

The design bending moments are equal to the maximum calculated flexural moments both in the span and over the supports. Reducing bending moments by chosing a critical section some distance from the centre of gravity of the support member is not permitted.

Simple Span Construction

For optimum design or analysis purposes, the design span of a simple span flexural member is the actual length of the member, measured centre to centre of the end connections, not the distance, centre to centre of the supports. The load eccentricity into the support (column) should be considered when span lengths centre to centre of connections are used. The reduction in bending moment is illustrated in example 1 of 4.05 when the actual span instead of the centre to centre distance between supporting members is used.

4.03 Analysis

Plastic vs. Elastic

Usually elastic methods are used for the analysis of an existing structure. Plastic analysis may be used provided certain criteria are met which are above and beyond those required for elastic analysis. These additional criteria for plastic design applicable to conventional building construction include:

1) Use of structural steel with $F_y \leq 80\ F_u$. This type of steel is normally found in ordinary structures.

2) Width-thickness ratios of the members should meet the requirements of those for plastic sections (class 1). Most, but not all, rolled wide flange beams meet these requirements.

3) Special bracing is required at plastic hinge locations. Such bracing is not commonly found in conventionally designed structures.

4) Web stiffeners are required at concentrated load locations where plastic hinges form. Such stiffeners are not commonly found in conventionally designed structures.

5) Splices in beams and columns must transmit the greater of $.25\ M_p$ or $1.1\ M_F$ at the splice location. Such splices are not common in buildings designed by elastic analysis.

Connections

During the design process, it is a common assumption that all connections act as frictionless hinges and to neglect the stiffening effect or the rigidity of connections. When re-analysing an existing structure, bending moments may be reduced, especially in simple span beams, if connection stiffness is considered in the analysis. For example, double web clip angle connections, top and seat plate connections, header plate connections or end plate connections all possess adequate fixity to induce partial fixed-end moments into simple span beams. The partial end fixity reduces the positive bending moment in the beam.

A procedure for calculating the end moments which can be developed in connections is given by Frye and Morris([1]). The procedure involves calculation of a moment rotation curve for the specific type of connection involved. The moment-rotation curve is then plotted M vsx $\emptyset$). The fixed end moment Mo (0 rotation) and the simple span rotations $\emptyset_s$ (0 moment) are plotted on the same graph and joined by a straight line. The intersection of the moment rotation curve and the straight line joining the fixed-end moment and simple end rotation limits is the bending moment which can be induced in the connection at the end of the beam. Moment rotation curves for steel beams with various end connections are shown in Figure 2.

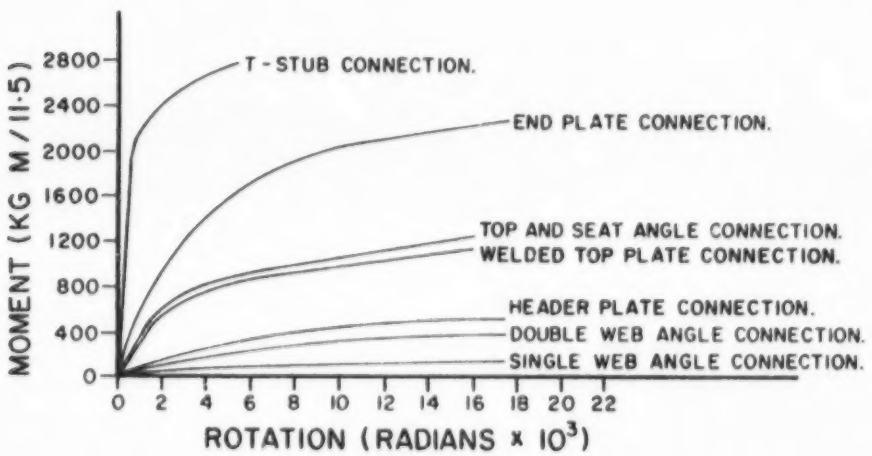


FIGURE 2 - Moment Rotation Curves for Steel Beams

The moment rotation functions for the several types of end connections are illustrated in Table 1.

CONNECTION TYPE	STANDARDIZED MOMENT ROTATION FUNCTION ($\emptyset$)	STANDARDIZATION CONSTANT (K)
Double Web Angle Connection	$3.66\,(kM) \times 10^{-4} + 1.15\,(kM)^{+3} \times 10^{-6} + 4.57\,(kM)^{+5} \times 10^{-8}$	$d^{-24}t^{-1.81}g^{0.15}$
Header Plate Connection	$5.10\,(kM) \times 10^{-5} + 6.20\,(kM)^{3} \times 10^{-10} + 2.40\,(kM)^{5} \times 10^{-13}$	$t^{-1.6}g^{1.6}d^{-2.3}w^{0.5}$
Top and Seat Angle Connection	$8.46\,(kM) \times 10^{-4} + 1.01\,(kM)^{3} \times 10^{-4} + 1.24\,(kM)^{5} \times 10^{-8}$	$t^{-0.5}d^{-1.5}f^{-1.1}l^{-0.7}$

TABLE 1 - Standardized Connection Moment Rotation Functions ([1])

The partial fixed end moments reduce the positive bending moment in the beam by their same magnitude. An example of the calculations involved is shown in section 4.05, example 2. It can be seen that reductions of up to 10% in the maximum, positive bending moment in the beam can be obtained by using this technique. The end moment in the beam must be distributed into the supporting column and must therefore be considered in the column analysis.

Continuity

Wherever possible, continuity should be assumed in the analysis of framed steel structures. A common type of construction, called semi-continuous or gerber-type, is often used for the roofs of the buildings. It is inexpensive to frame and efficient. It uses most of the benefits of continuous construction without imposing the costs of making moment resisting connections.

There are two types of members in gerber construction, the drop-in beams and the cantilever beams. The analysis of the drop-in beams or link-beams, as they are sometimes called, is quite straightforward. The span of the link beam is the centre to centre distance of the connection and the beam is considered to be simply supported with partial fixity at the end connection.

The cantilever span is somewhat more complicated because of the potential buckling of the cantiliver section. Normally, the length of the cantilever is limited to less than 15% of the main beam's span to prevent reduction in allowable bending stresses in the cantiliver portion. In practice, however, many structures have been constructed with cantilevers in excess of this 15% limit and the possibility of premature buckling failure of the cantilever should be examinied in detail. In a paper by Nixon and Adams([2]) the maximum overhang on cantilever beams, without reduction due to torsional buckling, is as follows:

$\lambda = .20 - \frac{ld/A_f}{31,000} \leq .16$ for interior girders

$\lambda = .22 - \frac{ld/A_f}{50,000} \leq .21$ for end girders

where, λ is the permissable fraction of the span length for the cantilever

l = span length

d = girder depth

A_f = area of girder flange

4.04 Code Changes

There have been many changes in the building codes and the steel codes since their inceptions. Only one change will be reviewed in this manual, one in which significant changes have occurred.

Member Resistance - Compression, Bending and Combined Bending and Compression

The codes have been continually modified to reflect ongoing research in the field of member resistance. Formulae have become more complex and more numerous in an attempt to establish a uniform safety coefficient for all types of members, but the basic considerations on which the resistance formulae have been derived, have not changed.

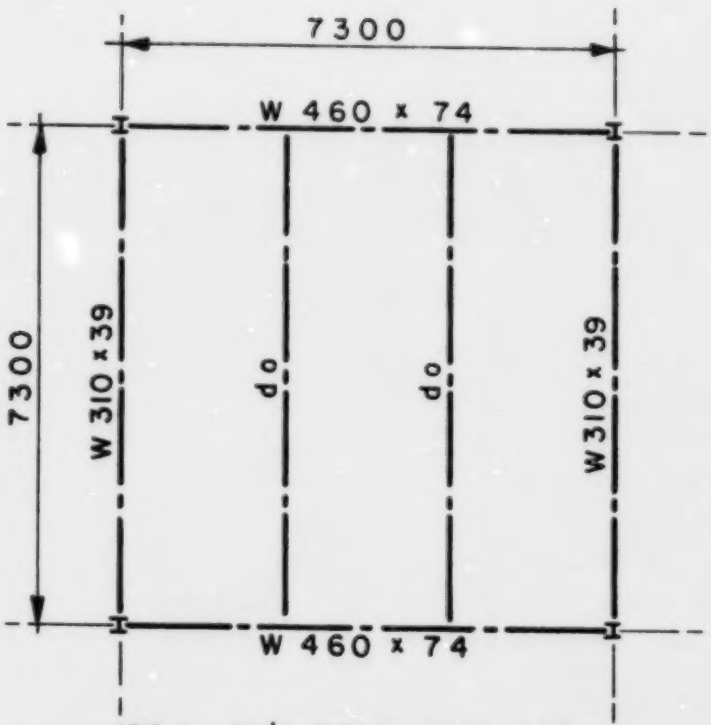
For compression, direct stress and torsional buckling are the significant factors which form the basis of the equation although evaluated by different equations.

For bending, direct stress and lateral torsional buckling equations still form the basis of the analysis.

For combined stresses, a unity type of equation based on direct stresses has always formed part of the resistance equations but more recently a stability equation has been added as well.

4.05 **Examples**

Example 1



DESIGN LOADS FROM DRAWINGS

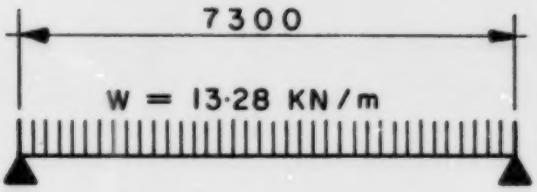
DEAD LOAD:

Conventional roofing	.34 MPa
100 mm slab	2.39 MPa
Steel	.19 MPa
Mechanical, electrical, ceiling	.24 MPa
	3.16 MPa

LIVE LOADS:

Snow	2.30 MPa
TOTAL	5.46 MPa

STEP 1 - Check of 1963 design - based on 1960 NPC
- simplified assumptions



$$M = \frac{wL^2}{8}$$

$$= \frac{13.28 \times 7.3^2}{8} \text{ kN.M}$$

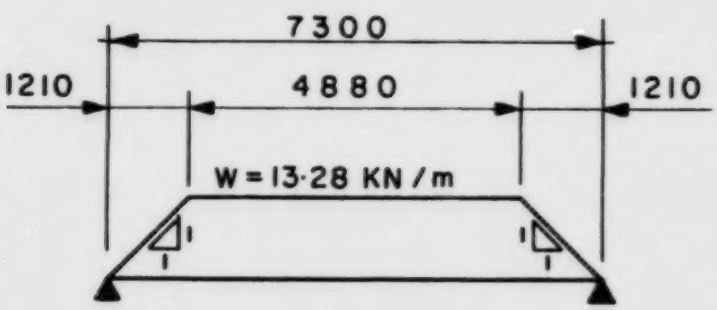
$$= 88.46 \text{ kN.M}$$

$$Sxx \text{ required} = \frac{M}{fs} \quad \text{where } fs = .61\ fy$$

$$= \frac{88.46}{.61 \times 248} \text{ m}^3$$

$$= .5847 \text{ m}^3 = 585 \times 10^3 \text{mm}^3$$

STEP 2 - Check of 1963 design - based on 1980 NBC
- detailed analysis



STEP 2 (cont'd)

$$R_L = R_R = \frac{13.28 \times 4.867}{2} + \frac{1.217 \times 13.28}{2}$$

$$= 40.40 \text{ kN}$$

$$M = \frac{40.4 \times 78.3}{2} - \frac{1.217 \times 13.28}{2} \times \left(\frac{4.867}{2} + \frac{1.217}{2}\right) + \left(\frac{4.867}{2}\right)^2 \times \frac{13.28}{2}$$

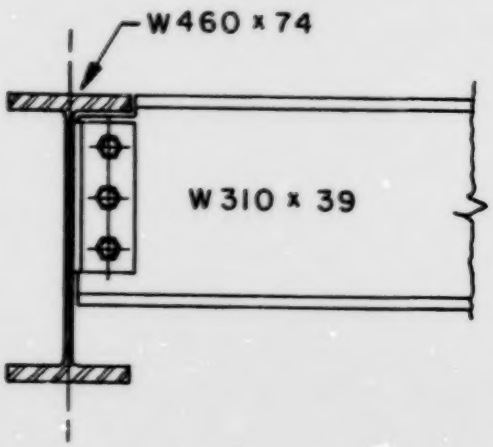
$$= 85.2 \text{ kN.M}$$

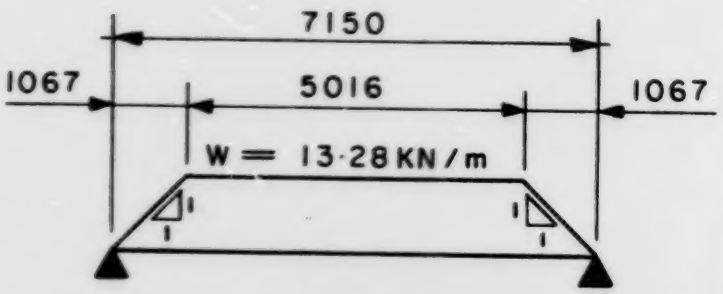
$$\text{Sxx required} = \frac{85.2}{.61 \times 248}$$

$$= 563 \text{ m}^2 = 563 \times 10^3 \text{mm}^3$$

Sxx provided = $549 \times 10^3 \text{mm}^3$

Consider revised span length - span length permitted to be centre/centre of connections
- using nominal double header connections this gives an overall span of 7150 mm





STEP 2 (cont'd)

$$R_L = R_R = \frac{13.28 \times 4.867}{2} + \frac{1.067 \times 13.28}{2}$$

$$= 39.40$$

$$M = \frac{39.40 \times 71.50}{2} - \left(\frac{4.867}{2}\right)^2 \times \frac{13.28}{2} - \frac{1.067 \times 13.28}{2} \times \left(\frac{4.867}{2} + \frac{1.067}{3}\right)$$

$$= 81.77$$

$$Sxx \text{ required} = \frac{81.77}{.61 \times 248}$$

$$= .540 \text{ m}^3 = 540 \times 10^3 \text{mm}^3$$

$$WTOT = 5.46 \times \frac{549}{540}$$

$$= 5.51 \text{ kN/m}^2$$

$$W_L = 5.51 - 3.16$$

$$= 2.35 \text{ kN/m}^2$$

STEP 3 - Analyzed by Limit States Design

$$fy = 248 \text{ MPa}$$

$$Mu = \phi \, fy \, Z$$

$$= .9 \times 248 \times .610$$

$$= 136.15 \text{ kN-M}$$

From previous calculation W & M

$$\frac{Wu}{W \text{ Design}} = \frac{Mu}{M \text{ Design}}$$

$$Wu = W \text{ Design} \times \frac{Mu}{M \text{ Design}}$$

$$= 5.46 \times \frac{136.15}{81.77}$$

$$= 9.09 \text{ kN/m}^2$$

STEP 3 (cont'd)

Load Factors: $\alpha_D = 1.25$

$\alpha_L = 1.50$

$W_{UD} = 1.25 \times 3.16 = 3.95 \text{ kN/m}^2$

$W_{UL} = W_U - W_{UD}$

$= 9.09 - 3.95$

$= 5.14 \text{ kN/m}^2$

$W_L = \frac{W_{UL}}{\alpha_L}$

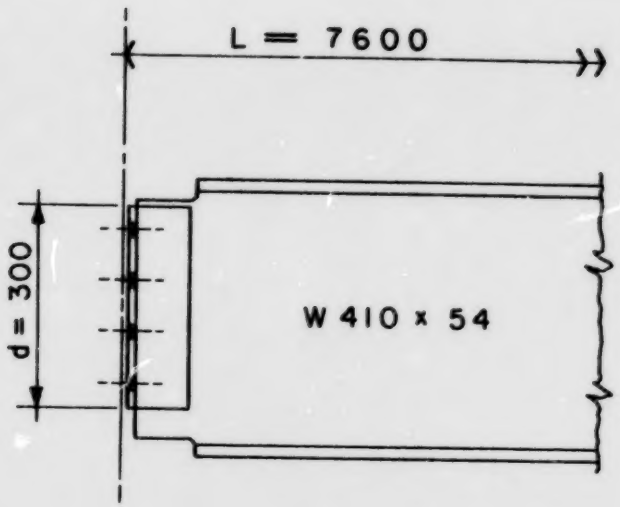
$= \frac{5.14}{1.5}$

$= 3.42 \text{ kN/m}^2$

CONCLUSION

By CSA S16.1 - Limit States Design there is a 1.12 kN/m^2 increase in e live load capacity over the 1960 design loads and a 1.07 kN/m^2 increas in live load capacity over the capacity analyzed under the 1960 NBC.

Example 2



RESTRAINT IN BEAM CONNECTIONS

Refer to "Analysis of Flexibly Connected Steel Frames" by M.J. Frye and G.A. Morris, Canadian Journal of Civil Engineering, Sept. 1975, pp. 285.

Note: Equations given in Imperial Units
Problem solved in Imperial Units

STEP 1 - For a Double Header Connection

$K = d^{-2.4} t^{-1} 81 g^{.15}$

for W410 x 54

$K = 12^{-2.4} \times .313^{-1.81} \times 3.75^{-.15}$

$= 0.256$

$\emptyset = 3.66\ KM \times 10^{-4} + 1.15\ (KM)^3 \times 10^{-6} + 4.57\ (KM)^5 \times 10^{-8}$

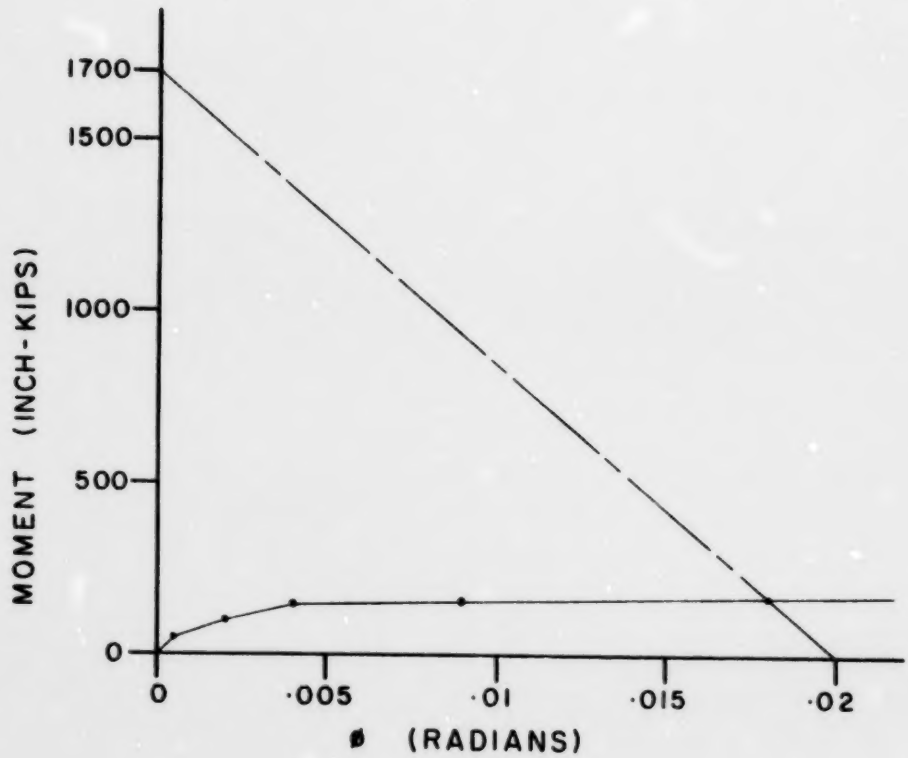
for W410 x 54

$\emptyset = 3.66 \times 0.256\ M \times 10^{-4} + 1.15 \times (.0256)^3 M^3 \times 10^{-6} + 4.57\ (.0256)^5 M^5 \times 10^{-8}$

$= 9.37 \times 10^{-6} M + 1.93 \times 10^{-11} M^3 + 1.03 \times 10^{-13} M^5$

PLOTTING THESE VALUES:

M (in-kip)	Ø (rad)	
0	.0	
50	.00050	
100	.00199	
150	.00929	
200	.0350	
400	1.0590	FAILURE



For W410 x 54

$M_R = Ø\ M_p$

$= .9 \times 44 \times 63.9$ in - k

$= 2530$ in - k

EQUIVALENT UDL

$$W = \frac{8\,MR}{L^2}$$

$$= \frac{8 \times 2530}{(25 \times 12)^2}$$

$$= .225 \frac{K}{in}$$

FIXED END MOMENT

$$M_F = \frac{WL^2}{12^2}$$

$$= .225 \times \frac{(25 \times 12)^2}{12} \text{ in - k}$$

$$= 1687 \text{ in - k}$$

ROTATION WITH FIXED END MOMENT

$$\text{Øs} = \frac{M_F\,L}{2E\,I}$$

$$= \frac{1687 \times 25 \times 12}{2 \times 29{,}000 \times 446}$$

$$= .0195 \text{ radians}$$

CENTRE-LINE BEAM MOMENT

$$M = \frac{WL^2}{8} - \text{end moment}$$

$$= 2530 - 160$$

$$= 2370$$

INCREASE IN CAPACITY

$$\frac{2530}{2370} = 1.06 \quad \text{say } 6\%$$

5.00 COMPOSITE CONSTRUCTION

As discussed in Chapter 5 - Detailed Analysis of Reinforced Concrete Structures, there have been significant changes to the Codes in connection with the design of composite steel/concrete beams.

5.01 Full Composite

Until the adoption of the 1965 National Building Code, composite action could be assumed to occur between the steel beam and concrete slab without full encasement of the beam and without the use of positive shear connectors, provided the shear stress between the steel flange and the concrete was less than .25 fc'.

In 1965 the requirements were changed. Now, either the has to be completely encased with 50 mm of reinforced concrete or shear connectors adequate to transfer the entire inter-material shear have to be installed on the beams.

5.02 Hollow Composite

In the CSA Code S-16 1969([4]), recognition of "hollow composite" action was first introduced. Hollow composite construction uses metal ribbed deck as formwork for composite construction. Certain criteria had to be met for the underside slabs profile:

1) minimum clear distance between ribs was 125 mm;
2) maximum rib height was less than 38 mm or 40% of the overall slab thickness;
3) average width of rib was more than 20% of the minimum clear distance between ribs or more than 25% of the overall slab thickness.

In the 1974 CSA Standard([4]), the requirements for the type of ribbed deck were relaxed somewhat to reflect the more common deck types

available. The allowable loads associated with shear connectors used with such deck were also redefined. The current CSA S16.1-M77([4]) is very similar to the 1974 code.

Any system which does not meet the current code requirements of either complete concrete encasement of the beam or use of shear connectors, should be analysed as non-composite construction.

6.00 REFERENCES

1. "Analysis of Flexibly Connected Steel Frames", M.J. Frye and G.A. Morris, Canadian Journal of Civil Engineering, Vol. 2, 1976, pp. 280 to 291.

2. "Design of Light Industrial Buildings", D. Nixon and P. Adams, Proceedings of Canadian Structural Engineering Conference, Toronto, 1978.

3. Handbook of Steel Construction, Third Edition, Canadian Institute of Steel Construction, 1980.

4. CSA Standards S16-1969, S16.1-1974 and S16.1-M77 Steel Structures for Building.

5. National Building Code of Canada 1948, 1953, 1960, 1965, 1970, 1975, 1980.

6. "Commentary G", The Supplement to the National Building Code of Canada 1980.

APPENDIX "A"

HISTORY OF DESIGN CODE CHANGES IN CANADA

1942 TO PRESENT

1942 NATIONAL BUILDING CODE

1.0 LOADS

1.1 Live Load:

- reduction factor of 15% to beams, girders and trusses when tributary area is 200 sq. ft. or more.
- reduction factors of up to 20% for storage and warehousing;
  - 30% for manufacturing, storage garage;
  - 50% other.
  - for columns, piers, walls and foundation walls supporting 8 floors or more.

1.2 Snow Load:

- roof slopes 20^{o} or less, 20 to 40 psf uniform snow load.
- roof slopes greater than 20^{o}.

  $L_1 = L(1 - 0.0233 (\alpha - 20))$

  where $L = S + R$

  S = average winter rainfall (inches)

  R = average winter snowfall (inches)
- general comments added regarding provision for snow accumulation where unusual snow depths occur.

2.0 CONCRETE STRUCTURES

2.1 Method - working stress design only permitted.

2.2 Minimum Reinforcement

- spacing of bars less than 5 times slab thickness or 24".
- shrinkage & temperature
- floor slabs - plain bars - .0025 bt
  - deformed bars - .002 bt
- roof slabs - plain bars - .003 bt
  - deformed bars - .0025 bt
- minimum flexural steel to be greater than minimum shrinkage and temperature steel.

2.3 Two-way Slabs

- separate design criteria for two-way slabs with beams or supports on 4 sides.

2.4 Flat Slabs

- an empirical method, similar to the "direct design" method permitted unless geometry of system irregular.
- provision for two-way (orthogonal) and four-way (orthogonal and diagonal) reinforcing schemes.

3.0 Steel Structures

3.1 Method

- working stress design only permitted.

3.2 Structural Steel

- basic allowable stress
- reduction in allowable stress to account for:
  bending - laterally unsupported beams
  columns - lateral torsional buckling.
- one unity equation for combined axial and bending stresses.

3.3 Steel Joists

- joists permitted only in conjunction with a concrete deck slab.
- maximum span/depth ratio of 24.
- maximum spacing 30".
- minimum slab thickness 2" for 24" spacing, 2½" for over 24" spacing.
- maximum panel spacing 12 times concrete slab thickness otherwise consider top chord combined axial and bending forces.
- bridging - top chord only required.
  - spacing at 7'0" centres or 40 times breadth of top chord.

3.4 Composite Construction

- permit either total encasement of beam or slab resting on unpainted top flange of steel beam.
- no shear connectors required if shear stress between steel and concrete less than .025fc'.
- shear connectors only required to carry excess shear in excess of .025 fc'.

1948 NATIONAL BUILDING CODE

1.0 Loads

1.1 Live Load - sames as 1942 NBC.

1.2 Snow Load - same as 1942 NBC.

2.0 Concrete Structures

2.1 Method - same as 1942 NBC.

2.2 Minimum Reinforcement - same as 1942 NBC.

2.3 Two-way Slabs - same as 1942 NBC.

2.4 Flat Slabs - same as 1942 NBC.

3.0 Steel Structures

3.1 Method - same as 1942 NBC.

3.2 Structural Steel - same as 1942 NBC.

3.3 Steel Joists - same as 1942 NBC.

3.4 Composite Construction - same as 1942 NBC.

1953 NATIONAL BUILDING CODE

1.0 Loads

1.1 Live Loads - same as 1942 NBC.

1.2 Snow Loads

- ground snow loads tabulated for various regions of Canada.
- roof slopes 20° or less, use full ground snow load
- roof slopes greater than 20°,

  $L_1 = L\ (1 - 0.0023\ (\ \ - 20))$

  where L = ground snow load

  where = roof slope
- general comments added regarding provision for snow accumulation or partial loading due to the effect of wind and the shape of the structure.

2.0 Concrete Structures

2.1 Method

- working stress design only permitted

2.2 Minimum Reinforcement

- spacing of bars less than 5 times slab thickness or 18".
- shrinkage and temperature reinforcement

floor slabs	- plain bars -	.0025 bt
	- deformed bars	.002 bt
roof slab	- plain bars	.0030 bt
	- deformed bars	.0025 bt

- mimimum flexural steel to be greater than the minimum shrinkage and temperature steel.

2.3 Two-way Slabs

- separate design criteria for two-way slabs with beams or supports on 4 sides.

2.4 Flat Slabs

- detailed provisions incorporated for "elastic analysis" method of flat slab design.
- minor differences in slab/column stiffness parameters and in distribution of steel to column and middle strips from current code requirements.
- "empirical method" (moment coefficient method) permitted similar to "direct design" method in current codes.
- four-way (orthogonal and diagonal) reinforcing scheme deleted.

3.0 Steel Structures

3.1 Method

- working stress design only permitted

3.2 Structural Steel

- first tentative provisions for high strength (A325) bolted connection introduced. Bolts to be designed as rivets.
- provisions introduced for moment redistribution in fully continuous beams and girders by permitting 20% overstress at interior supports.
- no recognition of "compact"sections but limits established for width/thickness ratios of projecting elements under compression.

3.3 Steel Joists

- maximum span/depth ratio of 24.
- spacing limited to 30" maximum unless joists designed as trusses.
- maximum panel spacing 12 times concrete slab thickness otherwise consider top chord combined axial and bending forces.
- bridging required to be able to reist a bending moment (in lbs.) equal to 250 x spacing of joist in inches.

3.4 Composite Construction

- same as 1942 NBC.

1960 NATIONAL BUILDING CODE

1.0 Loads

1.1 live Loads

- same as 1942 NBC.

1.2 Snow Loads

- climatic charts for various localties show ground snow load = roof snow load/.8
- ground snow loads X.8 applied to roof having slopes less than 30°.
- ground snow load reduced linearly by a coefficient varying from .8 to 0 for slopes from 30° to 70°.
- specific criteria introduced for drifting snow on canopies and on roof adjacent higher roofs. Such roofs required to be designed for 1.5 times gound snow load.
- specific requirements introduced for sliding snow from adjacent roof surfaces.
- unbalanced or drifting criteria for peaked roofs introduced.
- unbalanced loading criteria introduced for flat roofs.

2.0 Concrete Structures

2.1 Method

- working stress design only permitted.

2.2 Minimum Reinforcement

- spacing less than 3 time slab thickness or 15"
- shrinkage and temperature slabs with plain bars - .0025 bt
- slabs with deformed bars - .002 bt
- minimum flexural steel to be greater than the minimum shrinkage and temperature steel.

2.3 Two-way Slabs

- separate design criteria for two-way slabs with beams or supports on 4 sides.
- method substantially different from previous codes; moment coefficients introduced and continuity in slabs recognized.

2.4 Flat Slabs

- two methods permissable "empirical" (direct design) approach and elastic method.
- modifications to stiffness factors and to distibution of moments between column and middle strips.
- no specifc provision for transfer of unbalanced moments from slab to column in elastic analysis.

3.0 Steel Strutures

3.1 Method

- working stress design permitted.
- plastic design permitted.

3.2 Structural Steel

- slight increase in allowable stresses.
- high strength bolted connections specification finalized.

3.3 Steel Joists

- spacing limitation (30") removed from specifications for open web steel joists.
- specific provisions for top chord lateral support introduced; ie. slab or decking to be capable of resisting a lateral force of 20% of the vertical load on the joist.
- panel point spacing limited to 24" unless combined axial and bending effects considered on top chord.
- bridging - spacing at 7'0" centres.
  - slenderness ratio limited to 200.

3.4 Composite Construction

- same as 1942 NBC.

1965 NATIONAL BUILDING CODE

1.0 Loads

1.1 Live Loads

- introduced new live load reduction factors for storage, manufacturing, stores, garage and assembly areas over 900 sq. ft. of (.5 + 15/ A)
- introduced live load reduction factor for other occupancies of (.3 + 10/ A)

1.2 Snow Loads

- introduced concept of basic snow load coefficient C_S = .8 for flat roofs.
- coefficient applied to basic ground snow load.
- introduced snow load coefficient C_S = .6 for exposed roofs.
- introduced unbalanced snow loading conditions for various shaped roof(peaked, curved, arched, etc.) similar to current code requirements.
- refined requirement for snow build-up around projections or adjacent higher roofs to a maximum of 3 times the ground snow load.
- refined provisions for sliding snow.
- mantained unbalanced loading conditions for flat roof similar to 1960 NBC.

2.0 Concrete Structures

2.1 Method

- working stress design permitted
- ultimate strength design introduced
- load factors for ultimated strength design:

  dead load - 1.5

  live load - 1.8

2.2 Minimum Reinforcement

- spacing of reinforcement less than 3 times slab thickness or 15".
- shrinkage temperature reinforcement:

  plain bars - .0025 bt

  deformed bars - .002 bt
- separate requirements introduced for minimum flexural reinforcement of flexural members (except slabs) 200/fy.
- minimum flexural reinforcement for slabs to be the same as minimum shrinkage and temperature reinforcement.

2.3 Two-way Slabs

- similar to 1960 NBC.

2.4 Flat Slabs

- similar to 1960 NBC.

3.0 Steel Strutures

3.1 Method

- working stress design permitted.
- plastic design permitted.

3.2 Structural Steel

- introduction of compact section concept
- 10% stress increase permitted for flexure in compact sections.

3.3 Steel Joists

- similar to 1960 NBC

3.4 Composite Construction

- current provisions introduced, ie. beam either to be fully encased in concrete or shear connectors to be installed.
- entire shear force between steel beam and concrete slab to be transferred by shear connectors.

1970 NATIONAL BUILDING CODE

1.0 Loads

1.1 Live Loads

- similar to 1965 NBC.

1.2 Snow Loads

- similar to 1965 NBC and current code requirements.

2.0 Concrete Structures

2.1 Method

- working stress design permitted
- ultimate strength design permitted
- ultimate strength design load factors

  dead load - pre 1973 - 1.5

  post 1973 - 1.4

  live load - pre 1973 - 1.8

  post 1973 - 1.7

2.2 Minimum Reinforcement

- similar to 1960 NBC.

2.3 Two-way Slabs

- similar to 1960 NBC.

2.4 Flat Slabs

- similar to 1960 NBC.

3.0 Steel Strutures

3.1 Method

- working stress design permitted.
- plastic design permitted.
- limit states design introduced in 1974
- limit states design load factors
    - dead load - 1.25
    - live load - 1.50

3.2 Structural Steel

- similar to 1965 NBC.

3.3 Steel Joists

- similar to 1965 NBC
- new provisions introduced in 1975.

3.4 Composite Construction

- similar to 1965 NBC.

1975 AND 1980 NATIONAL BUILDING CODE

BASICALLY SIMILAR TO THE REQUIREMENTS OF THE 1970 NATIONAL BUILDING CODE

APPENDIX "B"

HISTORY OF MATERIAL PROPERTIES IN

CONSTRUCTION IN CANADA

1900 TO PRESENT

REINFORCING STEEL SPECIFICATION STRESSES

Dates	Specification	Remarks	Minimum Yield Strength
1938 - 1954	CSA G30-1938	Billet Steel Bars	
		(Structural Grade)	33 ksi
		(Intermediate Grade)	40 ksi
		(Hard Grade)	50 ksi
	GSA G31-1938	Rail Steel Bars	50 ksi
1954 - 1962	CSA G30.1-1954	Billet Steel Bars	
		(Structural Grade)	33 ksi
		(Intermediate Grade)	40 ksi
		(Hard Grade)	50 ksi
	CSA G30.2 - 1954	Rail Steel Bars	50 ksi
1962 - 1972	CSA G30.1 - 1967	Billet Steel Bars	
		(Intermediate Grade)	40 ksi
		(Hard Grade)	50 ksi
	CSA G30.2 - 1967	Rail Steel Bars	50 ksi
	CSA G30.8 - 1962	Axle Steel Bars	50 ksi
	CSA G30.10 - 1964	Billet Steel Bars with 60,000 psi Minimum Yield Point	60 ksi

REINFORCING STEEL SPECIFICATION STRESSES
(cont'd)

Dates	Specification	Remarks	Minimum Yield Strength
1972 - 1977	G30.12 - 1972	Billet Steel Bars for Concrete Reinforcement	
		(Grade 40)	40 ksi
		(Grade 50)	50 ksi
		(Grade 60)	60 ksi
		(Grade 75)	70 ksi
1977	CSA G30.12 - M1977	Billet Steel Bars for Concrete Reinforcement	
		(Grade 300)	300 Mpa
		(Grade 350)	350 Mpa
		(Grade 400)	400 Mpa

STRUCTURAL STEEL SPECIFICATION STRESSES

Dates	Specification	Remarks	Minimum Yield Strength
1900 - 1935	ASTM A7 (Bridges)	Rivet Steel	25 to 30 ksi
		Soft Steel	26 to 31 ksi
		Structural Steel	27 to 33 ksi
		Medium Steel	30 to 35 ksi
	ASTM A9 (Buildings)	Structural Steel	27 to 33 ksi
		Medium Steel	30 to 35 ksi
		Rivet Steel	23 to 28 ksi
1935 - 1959	CESA S39 - 1935	Mild Structural Steel	27 to 33 ksi
	CESA S40 - 1935	Medium Structural Steel	30 to 36 ksi
1950 - 1959	CSA G40.2 - 1950	Structural Rivet Steel	33 ksi
	CSA G40.3 - 1950	Mild Structural Steel	30 ksi
	CSA G40.4 - 1950	Medium Structural Steel	33 ksi
1959 - 1964	CSA G40.4 - 1959	Medium Structural Steel	33 ksi
	CSA G40.8 - 1960	Structural Steel with Improved Resistance to Brittle Fracture	* 40 ksi
			*(38 ksi)
			*(36 ksi)
1964 - 1971	CSA G40.8 - 1960	Structural Steel with Improved Resistance to Brittle Fracture	40 ksi
			*(36 ksi)
			*(38 ksi)
	CSA G40.12 - 1964	General Purpose Structural Steel	44 ksi
1971 - 1973	CSA G40.21 - 1971	Structural Quality Steel	44 ksi
			*(40 ksi)

STRUCTURAL STEEL SPECIFICATION STRESSES
(cont'd)

Dates	Specification	Remarks	Minimum Yield Strength
1973 - 1978	CSA G40.21 - 1973	Structural Quality Steel	
		(Grade 33 W)	** 33 ksi
		(Grade 38 W)	** 38 ksi
		(Grade 44 W)	44 ksi
		(Grade 50 W)	***50 ksi
1978	CSA G40.21M - 1978	Structural Quality Steel	
		(Grade 300 W)	300 MPa
		(Grade 350 W)	350 MPa

* Stresses reduced for thicker web or plate thicknesses

** Not commonly used for rolled steel structural shapes

*** Only commonly used for rectangular or square hollow structural sections